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Environmental Risk Assessment of Groundwater Recharge in Liwa Aquifer

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Environmental Risk Assessment of Groundwater Recharge in Liwa Aquifer

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Abstract

The Abu Dhabi Emirate has witnessed a remarkable development in the various aspects of life during the last three decades. Such rapid development imposes a tremendous pressure on natural resources including water. Conventional water resources in the Emirate are limited. The surface water is almost absent due to the scarcity and randomness of rainfall coupled with the prevailing arid conditions and high evaporation rates. In addition, groundwater is mostly brackish and non-renewable. Over-pumping practices have resulted into a severe decline in the groundwater levels and quality. A large portion of the freshwater demands in the Abu Dhabi Emirate is covered by desalinated water. Desalination plants often operate under a constant production capacity and are designed to meet the peak demands throughout the year. In other words, the freshwater production from the desalination plants remains constant regardless of the changes in demands not only from one season to the other but also during the same day. This has resulted in an excess of freshwater availability during specific periods where the demands are relatively low.

Water resources management and environmental issues are both addressed in a parallel manner in the Abu Dhabi Emirate. As a part of the water resources management plan, and to ensure the full utilization of the desalinated water, the excess of water during low demand periods can be used in recharging the depleting aquifers. This will help restore the groundwater resources and enhance the productivity of aquifers. However, environmental risk assessment of groundwater storage that could be contaminated from different sources should be conducted first to ensure the feasibility and effectiveness of this option. Groundwater contamination is mostly related to the construction of landfills near groundwater protective zones, spill of oil and other contaminants that might be released from gas stations, and vehicle accidents, pesticides contamination from farms, and lack of understanding of the consequences of such events.

The objective of this study is two-fold: first, to study the feasibility of artificial recharge of the groundwater resources in Emirate of Abu Dhabi, and second to employ a numerical groundwater model to simulate and predict the fate and transport of contaminants that might be released from different sources in the vicinity of recharge/discharge protection zones. Several scenarios, based on different assumptions, have been examined to study the potential impacts of contamination of groundwater around the wells of the Liwa aquifer located in the Western region of the Abu Dhabi Emirate. Through numerical modelling, the dimensions and location of the necessary groundwater protection zones are defined.

KEY WORDS: Aquifer storage/recovery, environmental risk assessment, groundwater artificial recharge, modelling, protection zone, wellhead protection area.

Chapter I

Introduction

1 Introduction

The United Arab Emirates is an arid country with no permanently natural surface water and limited groundwater resources. Under such conditions, artificial recharge of groundwater and storage of water in aquifers would play a major role in water resources management [Gale et al., 2002 and Abu-Taleb, 1993]. Due to increasing water demands and growing scarcity of reliable surface water resources in many countries around the globe, artificial recharge of groundwater and storing of freshwater in aquifers have been widely implemented to meet seasonal, long-term and crisis conditions in different countries [Moreland, 1998, Gale et al., 2002, Sadek and El Fadel, 1998]. The need of huge water storage systems to meet the demands during emergencies can not be over emphasized especially when desalination plants are at risk. Desalination plants might be exposed to natural disasters, industrial accidents and others.

The main source of freshwater in the Abu Dhabi Emirate is the desalinated water which is currently produced by five state-of–the-art plants. These plants cover the ever increasing freshwater demands for the urban supply [Sommariva and Syambabu, 2001]. Water availability and water demand are both subject to seasonal variations. Water demands may vary considerably not only seasonally but also daily. The Government of Abu Dhabi favours the option of storing the water surplus from desalination plants in specific aquifers like the aquifer of Liwa area for a period of up to five years. In arid areas, such as the Arabian Gulf region, where water demands exceed the available renewable water resources, freshwater from desalination plants is used to bridge this gap. To ensure water availability during emergencies, large freshwater storage capacities are required. Recharging aquifers with the excess of fresh water that are produced from desalination plants would provide a feasible solution.

1.1 Physical Setting of Abu Dhabi Emirate

Abu Dhabi is situated along the Arabian Gulf, between latitudes 22.5°, 25 ° North, and longitudes 51°, 55° East. It has an area of 67,340 km², which is equivalent to 86.7 % of the country's total area (Figure 1.1). Abu Dhabi City is the capital of the United Arab Emirates (UAE). The Emirate of Abu Dhabi comprises three major regions: Capital City Region (Abu Dhabi city and it environs), Eastern Region (Al-Ain city and its environs) and the Western and Central Region.

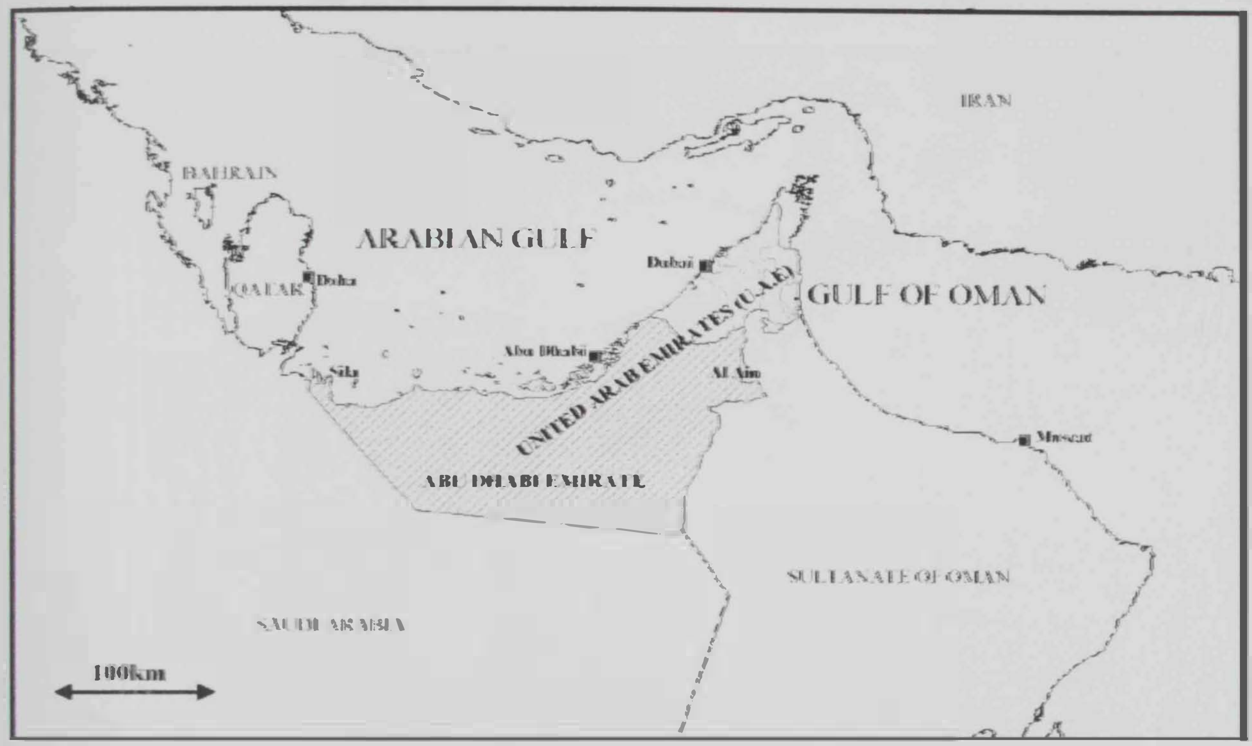

Figure 1.1 Location of Abu Dhabi Emirate

The land surface of Abu Dhabi Emirate is characterized by many natural geomorphologic settings. A simple classification of the geomorphology of the UAE has been made by the United States Geological Survey [USGS/NDS, 1993] in accordance with regions of hydrological significant, as shown in Figure 1.2. Three regions are covering the whole Emirate, Region I, Region II and Region III. Region I is the coastal Marine Zone stretching from As Sila to Gantout and including a total area of 13,400 km^2. This region is characterized by the natural tidal flats, sabkha coastal terrace and paleodunes. Sand and gravel aquifers are encountered in this zone and the groundwater in this region is very saline.

Region II is the largest among the three regions and comprises internal dunes and sabkha. Various surficial alluvial and sand aquifers are found in this region.

Region III is the Piedmont Plain covering an area of about 900 km^2 which is mostly alluvial fans and gravel plains. A large number of portable well fields are located in this region to benefit from the rapid recharge of surface water runoff in wadis and burrried alluvial channels. More details about the three different regions are presented in section 1.2.

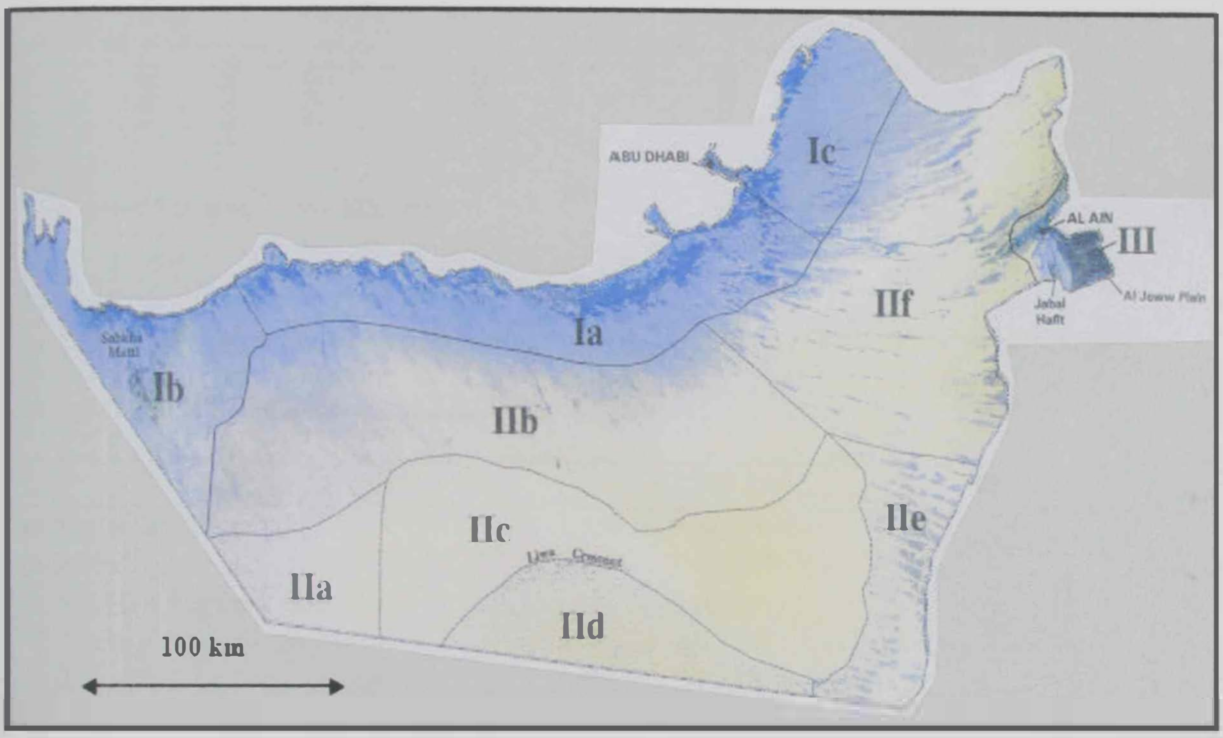

Figure 1.2 Physiographic regions of Abu Dhabi Emirate [EAD, 2005]

The climate in Abu Dhabi is characterized by its semi-arid to arid conditions with very high summer temperatures. The coastal area, where the bulk of the population is located, has a hot and humid climate in the summer from April to November [EAD, 2005] with a maximum temperature of almost 50°C, a daily average temperature of 35°C and the relative humidity often reaching 100% (Figure 1.3). In 2003, the mean daily sunshine in Abu Dhabi ranged from around 8 to 11 h/d [MOP, 2003]. The mean maximum wind speed ranged from 11 km/hr in January to 16 km/hr in April. Very little amount of rainfall occurs in summer. The general information about the Emirates is presented in Table 1.1. It should be noted, however, that the annual rainfall varies significantly from one year to another.

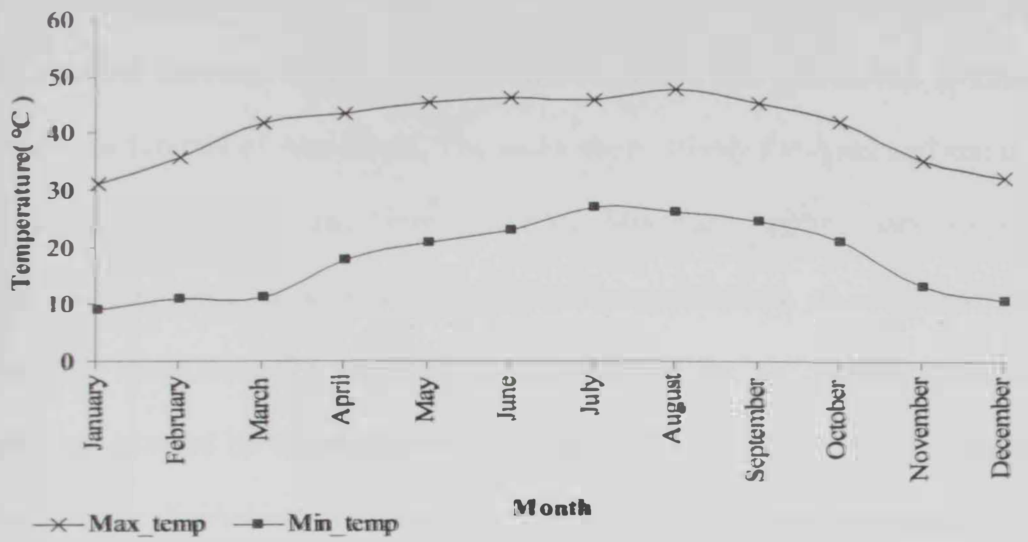

Figure 1.3 Maximum and minimum temperatures, 2003 at Abu Dhabi airport [MOP, 2003]

Table 1.1 General Information on the Abu Dhabi Emirate

Area	**67 340 km^2**
Maximum Elevation Point	1,163 m amsl, Jebel Hafeet
Population*	
Western Region - Abu Dhabi	858,650
Eastern Region - Al Ain	422,340
Islands	11,129
Total	1,292,119
Annual Population Growth 2001-5*	3.70%
Annual Rainfall, 2005**	
Abu Dhabi City	20.4 mm
Al Ain City (Airport))	33.8 mm
Annual Temperatures** (min/max)	
Abu Dhabi City	10.6 / 47.4 °C
Al Ain City	8.8 / 48.8 °C

* EAD, 2005 ** MOC, 2004

1.2 Geology of Abu Dhabi

The surface geology of Abu Dhabi is concealed under a cover of sands, which form dune ridges reaching heights of about 150 m inland. The coastal plains are dominated by tidal flats, sabkhas, which extend for more than 80 km southwards into the sand deserts [Brook et al., 2006]. Several thousand metres of consolidated marine and continental sedimentary rocks underline the Emirate of Abu Dhabi. The rocks are relatively flat-lying and aerially extensive beneath most of the Emirate. Near the Oman Mountains, sedimentary rocks are folded, faulted, and up-lifted in a complex pattern. Unconsolidated alluvial deposits cover the sedimentary rocks near the mountain front. Much of the sedimentary rocks and alluvial deposits are covered by unconsolidated sand dunes [Pallas, 1993]. The hydrogeology of the Emirate may be divided into two main parts; vertical and horizontal hydrogeology units.

1.2.1 Vertical hydrogeologic units

The vertical hydrogeological system can be subdivided into the following main units:

1. ***Top Shallow Aquifer (Water Table)***

This aquifer represents a natural water table aquifer and comprises all permeable layers that are hydraulically connected and exhibits a hydraulic head of water table. The main features of the shallow groundwater could be described from the groundwater flow systems which are controlled by many factors such as the recharge process, the geology of the host rock and the discharge process [EAD, 2005]. Figure 1.4 shows the three types of flow systems that occur within the Emirate; local, intermediate and regional.

Local system; occurs as springs, shallow hand dug wells, aflaj, etc. This system is limited to the eastern region, close to Oman border and is characterized by low salinity and a temperature close to ambient air temperature [Brook et al., 2006].

Intermediate system; Inland sabkha is the main discharge area and the groundwater contained in this system is encountered in relatively thin sand aquifers.

Regional system; Groundwater in this system is characterized by its low velocity. Long residence groundwater bodies move towards the North West, the Gulf and also to the South West into Saudi Arabia where the discharge areas encompass the low lying sabkha.

2. ***Bottom of Upper Aquifer***

The upper aquifer comprises the upper part to the shallow aquifer. The bottom part of the shallow aquifer can often be considered as an aquitard with a relatively lower hydraulic conductivity. In some area, the lower confining layer may be close to the surface [GTZ-DCO/ADNOC, 2005].

3. ***Lower Fars Formation / Aquiclude***

The lower Fars formation consists predominantly of mudstones with intercalated layers of dolomitic marlstones and evaporates. It underlies the quaternary sediments under most of the Western Region [GTZ-DCO/ADNOC, 2005].

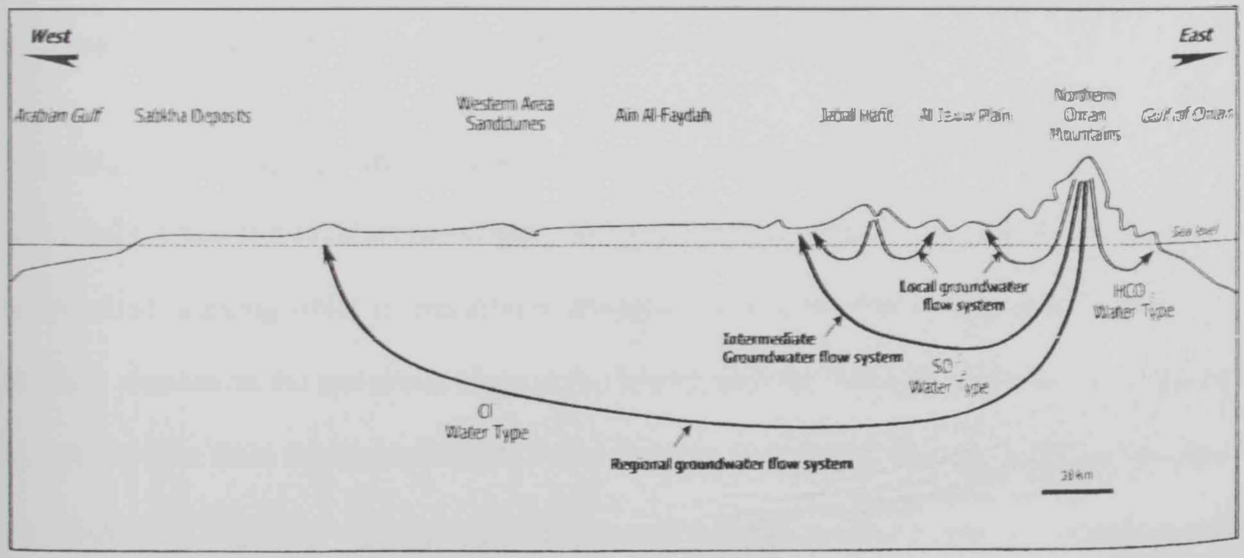

Figure 1.4 Regional groundwater flow systems [Alsharhan et al., 2001]

1.2.2 Horizontal hydrogeologic units

An overview of the areal hydrological units in the Emirate is shown in Figure 1.5 where the Emirate is divided into seven units. A brief description is given hereafter:

SA_L; This unit refers to shallow aquifer that covers most of the western region. It represents the Quaternary aquifer and the Western aquitard. The aquifer consists mostly of eolian sands and is directly underline by the lower Fars Formation as basal unit [GTZ-DCO/ADNOC, 2005].

SA_U; This unit represents the quaternary sand and gravel that is confined to the bottom by the Upper Fars Formation.

SA_S; This unit defines the coastal and inland sabkha area.

SA_M; This unit represents the shallow sand and gravel aquifer.

SA_J; This unit comprises the shallow sand and gravel aquifer of Al Jaww plain, situated to the east of Jebel Hafit.

Tm_B; This unit comprises Baynunah formation, which consists of Upper Miocenc fluvial sandstones with some conglomeratic layers.

Tm_F; This unit represents areas where the Lower Fars Formation, the main basal aquiclude of the Shallow aquifer in the Western Region are found close to the surface.

1.3 Water Resources in the Emirate of Abu Dhabi

Abu Dhabi Island is a sabkha coastal area, where the groundwater is very saline and therefore the required drinking water is exclusively produced by desalination of seawater. Al Ain city, which is situated on the mainland, close to the border with the Sultanate of Oman, is supplied with freshwater from desalinated water. In the Western and Central Regions, south of the Abu Dhabi–Al Ain highway, villages and settlements on the mainland and on islands were historically supplied by inland well fields. However, with decreasing supplies from these well fields, desalination plants have recently been introduced on the coast. In the area toward the Dubai border, villages and settlement on islands, the coastal zone and inland are supplied either with water from groundwater wells or from desalination plants.

The Emirate water demand is covered by groundwater, desalinated water and treated water. Groundwater contribution to the total water demand is about 71.2%. However, groundwater supply is decreasing and the imbalance between supply and demand is bridged by the ever increasing production of desalinated water (24% of the total demand). Treated wastewater contributes by 4.8% to the total demand and is totally consumed by the agricultural sector [EAD, 2006]. Figure 1.6 illustrates the percentage of each water resource in the Emirate. Main water resources in the emirate may be summarized as follows:

1.3.1 Rainfall

Participation within Abu Dhabi Emirate is erratic both in time and space. It provides the source of water for surface runoff which eventually recharge the aquifers, especially in Eastern Region of the Emirate where many wadi systems exist. Whilst this water recharge is very important, it contributes, on average, only 4 % of per annum of the total water consumption in the Emirate [EAD, 2005].

Most of the rainfall occurs during the winter months (December to March). February is on average the wettest month, with a mean of 30.7mm, although the yearly variability has ranged from null to 202.3 mm in the last 19 years [Abu Dhabi international Airport, 2005]. January is the coldest month, with an average temperature of 18°C. Spring and summer witness occasional concentrated heavy rainfalls. The rainfall distribution in the Emirate is highly variable over space and time as shown in Figure 1.7. For example, in 2003, the mean rainfall per month was about 2.9 mm (January), 0.2 mm (February), 3.9 mm (March) and 44.7 mm (April), respectively, while there was no precipitation in the remaining months [MOP, 2003]. For the Western Region, the mean annual rainfall is less than 50 mm/yr, while in the Eastern Region it varies from 80-100 mm/yr [EAD, 2005].

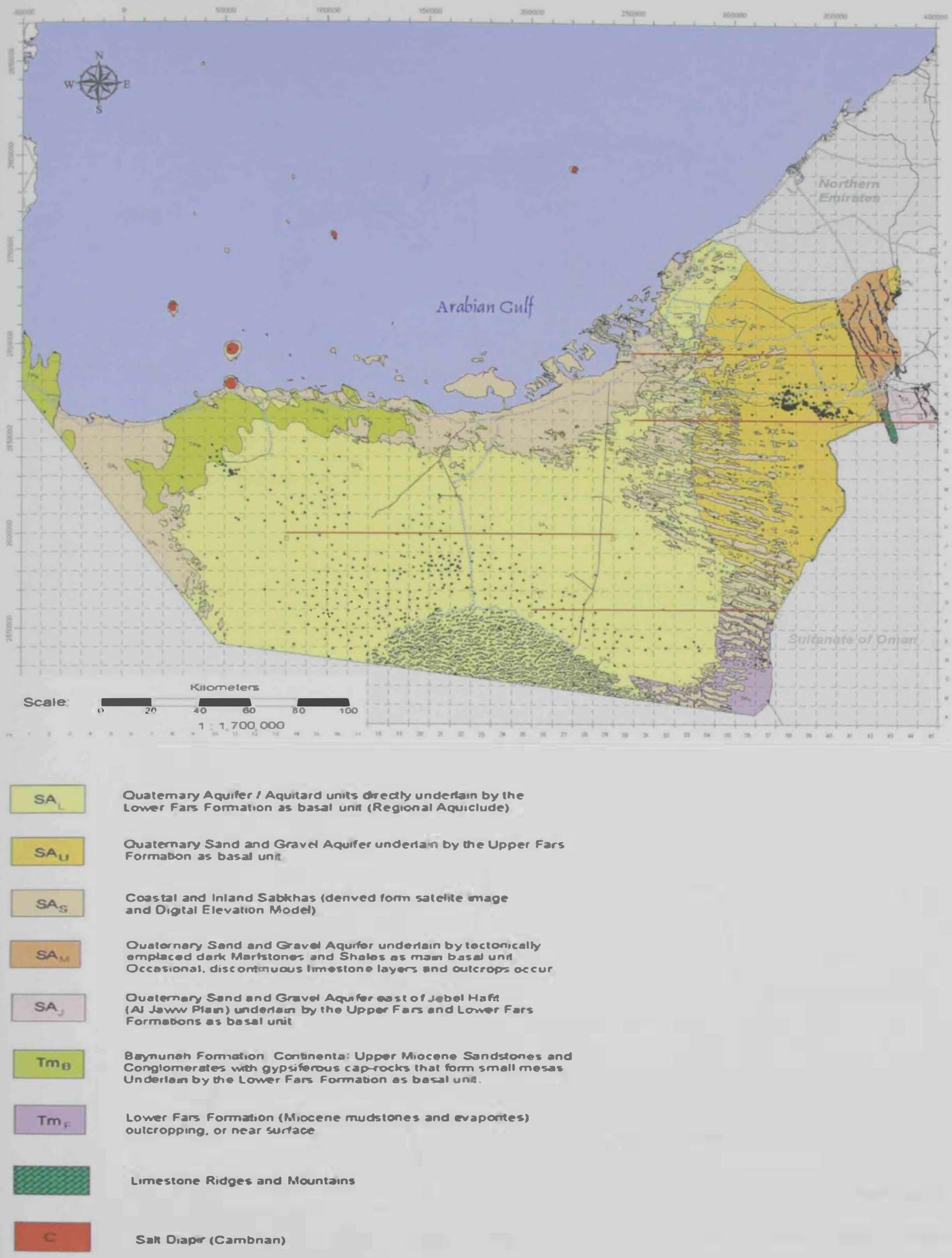

Figure 1.5 Hydrogeology overview of Abu Dhabi Emirate [GTZ-DCO/ADNOC, 2005]

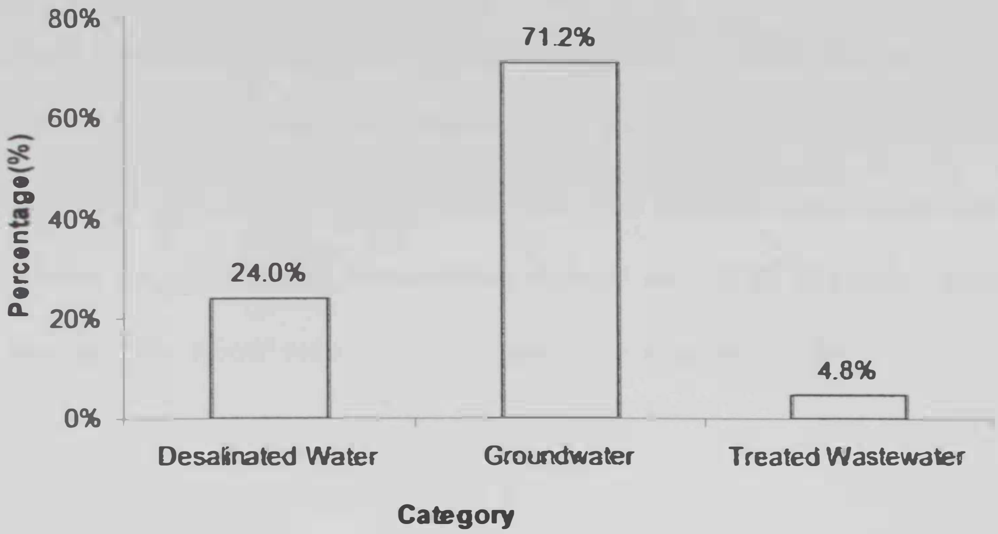

Figure 1.6 Abu Dhabi Emirate water resources [EAD, 2006]

Figure 1.7 Average rainfall in Abu Dhabi Emirate between 1990 and 2001 [MOC, 2004]

1.3.2 Groundwater

Approximately 50,000 wells exist in the Emirate of Abu Dhabi. Most of them are located in the agricultural hotspots of the Emirate (Al-Ain, Al Khazna-As Saad, Siwehan-Al Hiyar, Al Wijan and the Greater Liwa area). The estimated total average abstraction from these wells in 1994 was 0.31 Mm^3/d of freshwater and 0.71 Mm^3/d of brackish water [Moreland, 1998]. Table 1.2 shows the groundwater consumption in 2003 and 2005. The table indicates that there has been an 18% overall reduction in groundwater supply since 2003.

Table 1.2 Groundwater uses in the Emirate, Mm^3 [EAD, 2006]

	2003	2005	% Change
	Groundwater Uses, Mm^3		
Municipal	25.781	12.26	-52
Aflaj	5	5	0
Forestry	607.3	362.38	-40
Agriculture	1949.36	1741.43	-11
Amenity	114.19	104.85	-8
Total	2701.63	2225.92	-18

Most of the groundwater in the Emirate is found in the unconsolidated dune sand aquifers, alluvial deposits, shallow sedimentary formations, and shallow fractured/karstified limestone formation. Fractured limestone and uncemented alluvial gravel deposits are the most prolific sources of groundwater in the Eastern Region of the Emirate, while most of the groundwater resource in the Western Region is found in dune sand aquifers. About 1,440 km^2 in Al Ain area and 2,400 km^2 in the Greater Liwa area are underlain by fresh groundwater [GTZ-DCO/ADNOC, 2002]. Fresh groundwater mostly occurs in the shallow part of the aquifers, while the salinity of the deeper-seated groundwater increases significantly with depth, ranging from slightly brackish to highly saline. The imbalance between natural recharge and groundwater pumpage has resulted in a local groundwater level decline of more than 100 m since 1990 west of Al Ain [Moreland, 1998]. Abu Dhabi Emirate has been aggressive in planting forests. The forest area has grown steadily over the years. In 1994, about 201 Mm^3

of brackish groundwater was pumped to irrigate forest plantings throughout the Emirate [Moreland, 1998]. A small amount of reclaimed wastewater used to irrigate forests and landscaping. Farm and garden irrigation represents the main consumption element of groundwater in Abu Dhabi Emirate. Private and public wells pumped about 348 Mm^3 of brackish groundwater to irrigate farms. About 60 Mm^3 of fresh groundwater was also pumped [Moreland, 1998]. Wells completed in fractured limestone produce as much as 350 m^3/h [GTZ-DCO/ADNOC, 2003] and wells completed in semi-consolidated alluvial graved produce up to 400 m^3/h. Some wells completed in sedimentary formations in Al Khazna/Remah area produce more than 150 m^3/h. In general, fresh groundwater can be found in shallow aquifers in the recharge area near Al Ain and beneath the sand dunes north of the Liwa Crescent. Groundwater contains different concentrations of chemical constituents dissolved from host rock or introduced through man's activities. Table 1.3 shows the classification of groundwater based on the concentration of the total dissolved solids and its use in general. The groundwater reserve in the Emirate is estimated as 641 km^3 [EAD, 2006]. Less than 3% of the reserve is fresh and around 80% is saline water (Figure 1.8). Most of the groundwater reserve exists in the Western Region of the Emirate and contributes around 75 % of the fresh water. However, based on current abstraction rates in the Emirate, both fresh and brackish reserves will be depleted within the next 50 years [EAD, 2006].

Table 1.3 Classification of groundwater and its uses in Abu Dhabi Emirate [GTZ-DCO/ADNOC, 2003]

Type of Groundwater		**Concentration (mg/l)**	**Uses**
	Fresh	<1,500	Drinking livestock watering Irrigation of salt sensitive crops
Brackish	Slightly	1,500-4,000	Irrigation, desalination
	Medium	>4,000-7,000	Irrigation of salt tolerant crops, bushes, …
	Strongly	>7,000-10,000	
	Saline	>10,000-100,000	Pressurize (water flood) oil fields Source of water for Desalination plants
	Brine	>100,000	No use

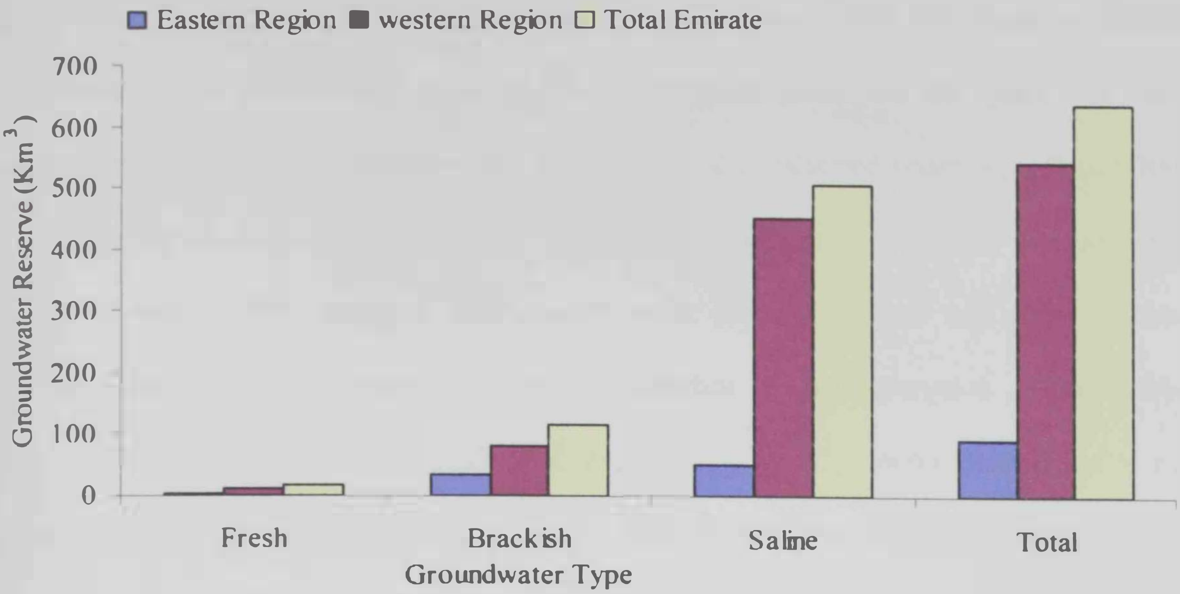

Figure 1.8 Groundwater reserve in Abu Dhabi Emirate [EAD, 2006]

1.3.3 Desalinated water

The conventional water resources in Abu Dhabi Emirate do not meet the national water demands. Therefore, the non conventional resources are intensively used nowadays, especially desalinated water. The remarkable economic and demographic developments of the UAE are closely associated with desalination technology; where 98% of the water supply comes from desalination of seawater or brackish water [Sommariva and Syambabu, 2001].

Figure 1.9 shows the distribution of desalination capacity in the UAE by emirates in which 61% of the total capacity is in Abu Dhabi Emirate. Last statistics of Abu Dhabi Water and Electricity Company (ADWEC) revealed that water produced from the desalination plant increases annually, and the dependence on the groundwater source decreases due to the depletion of aquifers. This confirms that desalination is the central source of fresh water in UAE, especially when the per capita water demand increases sharply [ADWEC, 2004].

Desalinisation plants in Abu Dhabi Emirate are composed of Multi Stage Flash (MSF), Multi Effect Distillers (MED) and Reverse Osmosis (RO) plants. Some of the RO plants are utilized to desalinate saline groundwater as in Al-Ain. In order to meet both the quantitative and qualitative requirements of drinking water, the majority of desalinated water is produced from MSF plants due to their integrity with power generation and their ability to produce large volumes of water. MSF produces high quality water (between 2 and 150 mg/l of total dissolved solids). The total quantity of water production from desalination plants in Abu Dhabi in 2005 was about 615 MGD (850 Mm^3/y) [ADWEC, 2005]. History of water production in the Emirate is given in Figure 1.10. Today, the evaporation technique is dominant in the desalination field, where 96% of desalinated water is produced by Multi Stage Flash (MSF) and Multi Effect Distillation (MED). The remaining 4% is produced by reverse osmosis. To improve the economics of desalination process, the MSF process is usually coupled with electric power generation in the so-called cogeneration plants. Table 1.4 lists the desalination plants in the Abu Dhabi Emirate with each plant capacity and the technology used. The majority of desalinated water in Abu Dhabi Emirate is produced from Arabia and Taweelah complexes plants, while the remaining is produced from desalination plants in Mirfa, Shuweihat and small plants in Sila, Dalma Island, Jabal Dhanaha and Sir Baniyas Island. The desalination units in remote areas are also shown in Table 1.5.

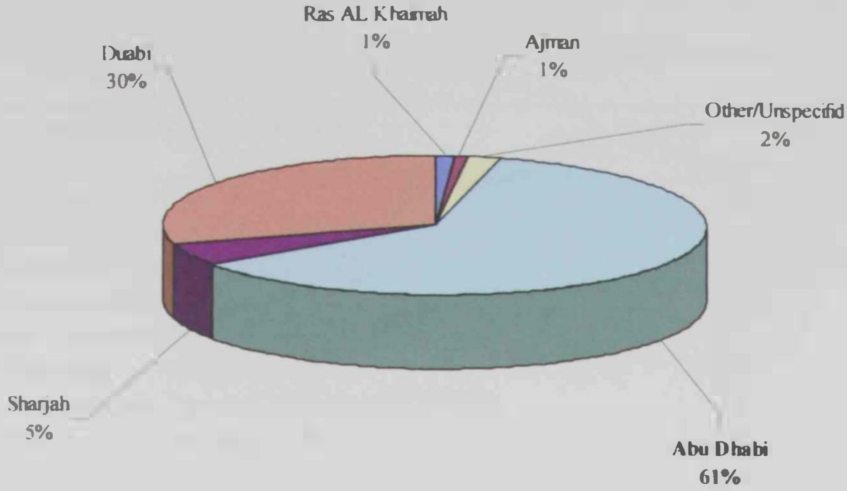

Figure 1.9 Distribution of desalination capacity in the United Arab Emirates, by Emirate [Wangnick, 2000]

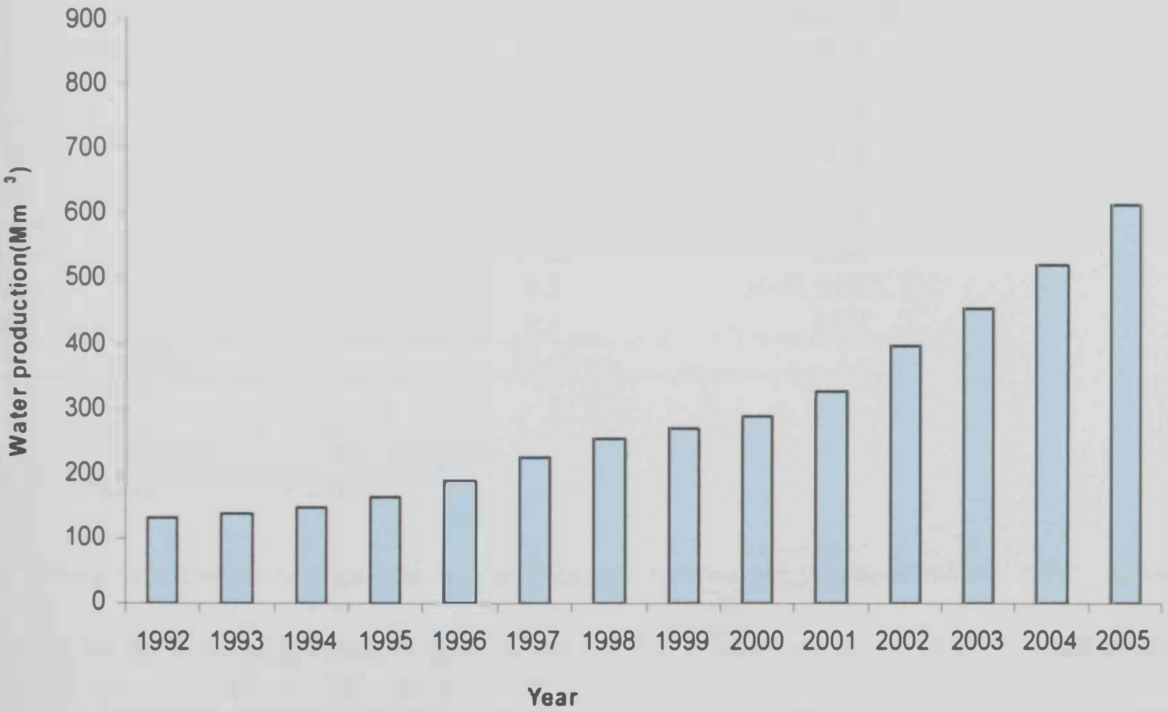

Figure 1.10 Production of desalinated seawater in Abu Dhabi Emirate [ADWEC, 2005]

Table 1.4 Water capacities of different cogeneration plants in Abu Dhabi Emirate

Desalination / Power Plant	Installed Capacity MIGD	Desalination Technology
Arabian Power Company (APC)	160	MSF, MED
Gulf total Tractable Power Company (GTTTPC)	84	MSF, MED
Emirates CMS Power Company (ECPC)	50	MSF
Bainohan Power Company (BPC)	15	MSF
Al Taweelah Power Company (ATPC)	100	MSF
Al Mirfa Power Company (AMPC)	100	MSF
Shuweihat CMS International Power Company (SCIPCO)	39	MSF
Union Water and Electricity Company (UWEC)	100	MSF, RO
Total	648	

Table 1.5 ADWEA's desalination units in remote areas

Site	Installed Capacity MIGD	Desalination Technology
Sila	2.25	MED, RO
Dalma	3	MED, RO
Jabal Dhanna	7	MED, RO
Barj Rumaith	0.5	MSF
Mirfa	2	MED, RO
Al Ariam	0.25	RO
Rafeeq	0.25	RO
Dabayah	0.36	MSF
Guzzlan	0.22	MSF
Marawha	0.22	MSF
Saadiat	0.3	MSF, MED, RO
Jemin	0.1	MSF
Total	16.45	

1.3.4 Treated wastewater

The treated wastewater is regarded as a new non-conventional source of water in Abu Dhabi that can be used to supplement a portion of needed water for agriculture and landscaping purposes. Concerns towards this type of water came as a consequence of immense efforts done by concerned authorities in the Emirate regarding reserving water resources and protecting the environment. Several benefits could result from using this source of water. By 2003, the population of Abu Dhabi Emirates reached 1.2 million and a total of 140.8 Mm^3 of treated wastewater was produced from 23 operating treatment plants in the Emirates [EAD, 2005]. The UAE, with specific reference to Abu Dhabi Emirate, is implementing the technology of recycling sewage water and using it in the proper way to develop green areas and parks in the cities. The use of the treated wastewater is conducted in an effective manner that would also account for environmental considerations [Al-Zubari, 1998]. The recycled/municipal wastewater is under the authority of Abu Dhabi and Al-Ain. It is produced and used for greenery (not farming) in the whole Emirate.

The feasibility of the treatment of sewage water and its re-use depends on many factors, such as the cost of treatment and the degree of required treatment in comparison to the cost of producing alternative water resource for the same purpose. As a matter of strategic management of using this source, treated wastewater is regarded as one of the solutions to decrease the stress on the natural water resources. The advantages of using treated wastewater include:

1. Protecting the environment from pollution. Wastewater will be produced irrespective of whether or not it is used. Treatment is essential from the environmental point of view.

2. Provide a dependable non-conventional water resource in Abu Dhabi to sustain the existing freshwater resources. As agricultural water demands and environmental needs grow, recycled water will play a greater role in the overall water supply.

3. Reusing treated water provides better option than its disposal from the economic and environmental points of view. Most of the recycled water in Abu Dhabi Emirate is used when there is a high water demand; fractional portions of unused treated wastewater are discharged to the sea.

Despite the above mention benefits, questions still remain about the possible use of treated sewage water in agricultural irrigation, and chemical and biological effects on the plants, soils and groundwater, in addition to the possible health hazards associated with its use. However, it is possible to avoid such consequences by safe treatment, which ensures the elimination of pollutants and purification of the water to be used for crop irrigation. It is essential to educate the community appropriately to avoid any adverse effects on the environment [Rizk, 1999]. Public awareness and knowledge regarding various aspects of treated wastewater are limited. This resulted in a generally negative public attitude toward various uses of the reclaimed water. Such negative attitude towards treated wastewater could be overcomed by educating the community and designing programs to encourage the reuse of the treated wastewater. Positive image can also be developed by using positive terminology, e.g. water recycling instead reuse of effluent reuse, recycled water instead reuse of treated sewage effluent, and so on.

Mafraq Wastewater Treatment Plant

The Mafraq Wastewater Treatment plant has been installed to receive and treat the sewage from Abu Dhabi city and its surroundings (Photo 1.1). It is located approximately 40 km from Abu Dhabi and is designed to handle a daily flow of 180,000 m^3/d (47.5 MGD). The plant can handle a peak flow of 260,000 m^3/d (68.6MGD). It employs tertiary treatment which produces water with high quality. The treated water is reused for irrigation of forest and development of green areas and landscaping.

The plant receives both industrial and the municipal wastes. The sewage undergoes complex biological treatments to free the water from bacteria [ERWDA, 2003] and provide high quality effluent to be used for irrigation around the city of Abu Dhabi. The final effluent is also used to irrigate a green oasis around the plant which is now a home to a wealth of wildlife [Biwater, 2005]. Successful example of the using treated water in Abu Dhabi is Al Wathba Wetlands Reserve, which attracts thousands of migrant waders and other waterfowl during the winter, and has also been the site for the first recorded breeding anywhere on the mainland of the Arabian Peninsula of the Greater Flamingo in the last 70 years. The lake's success is due to the use of recycled water from the Mafraq Sewage Treatment Plant. A substantial quantity of the treated waste water is used to irrigate fodder fields at the nearby Al Wathba Camel Track [ERWDA, 2004].

1.4 Water Storage Requirements

Total annual water consumption in the Eastern and Central regions of Abu Dhabi Emirate for 2005 stands at 3,111 Mm^3, of which 1,741 Mm^3 (56 %) of this water is used for agriculture irrigation. Some 363 Mm^3 (12%) of this water is used for forestry while 255 Mm^3 (8%) goes to amenity planting such as gardens, parks and roadside landscaping, and 677 Mm^3 (22 %) is consumed by domestic sector [EAD, 2006]. Of the total water consumption, about 71 % comes from groundwater.

Despite the shortage of water in the Emirate, water continues to be used unwisely, wasted and sometimes polluted. There is a lack of measures- policies -procedures and public incentives for water conservation. Inadequate attention is paid to the amount of water used by domestic sector and public buildings like mosques and schools. In addition, groundwater is becoming more saline and the salinity of the agricultural land has increased. More actions toward storing water for long term and during the minimum water demands are required.

The application of Aquifer Storage and Recovery (ASR) in Abu Dhabi Emirate is a potential solution to the water shortage problem in the Emirate. Desalinization plants are threatened by contamination from environmental disasters or other crises. For example in January 1998, the desalinization plants at Sharjah and Ajman in the Northern Emirates were shut down when an oil barge spilled nearly 5,000 tons of fuel oil in the Arabian Gulf [Hutchinson, 1998]. Because backup facilities were insufficient to meet water demands, many sectors experienced a water shortage. It might have been feasible to pump water from aquifers to satisfy the critical needs if a groundwater reserve had been established through implementation of an ASR program. Figure 1.11 shows the number of days of the available water storage in the

GCC countries. UAE has only 2 days storage capacity to meet the required demand. In addressing water storage requirements, it is important to consider the source of the water, existing storage capacity, the total demand and seasonal demand variations.

Abu Dhabi Water and Electricity Authority (ADWEA), which is responsible for providing water and electricity for the Emirate, uses many desalination techniques to produce water including Multi Effect Distillers (MED), Reverse Osmosis (RO) and Multi Stage Flash (MSF). The main technology that is used is the thermal desalination, MSF as it is more efficient and produces high quality water (2 - 150 mg/l TDS) in large quantities. It also has a low risk of bacterial or pathogenic contamination. At each desalination plant, there are water storage tanks for back-up use. The size of potable fresh water tanks at the desalination plants varies from 0.2-0.4 Mm^3. The total amount of storage at all plants is 1.51 Mm^3 which is less than a one day production as illustrated in Table 1.6. ADWEA has a water shortage that would meet the demands of 2 days. A crisis water management scenario was taken into consideration when dealing with the water management as described in the following section.

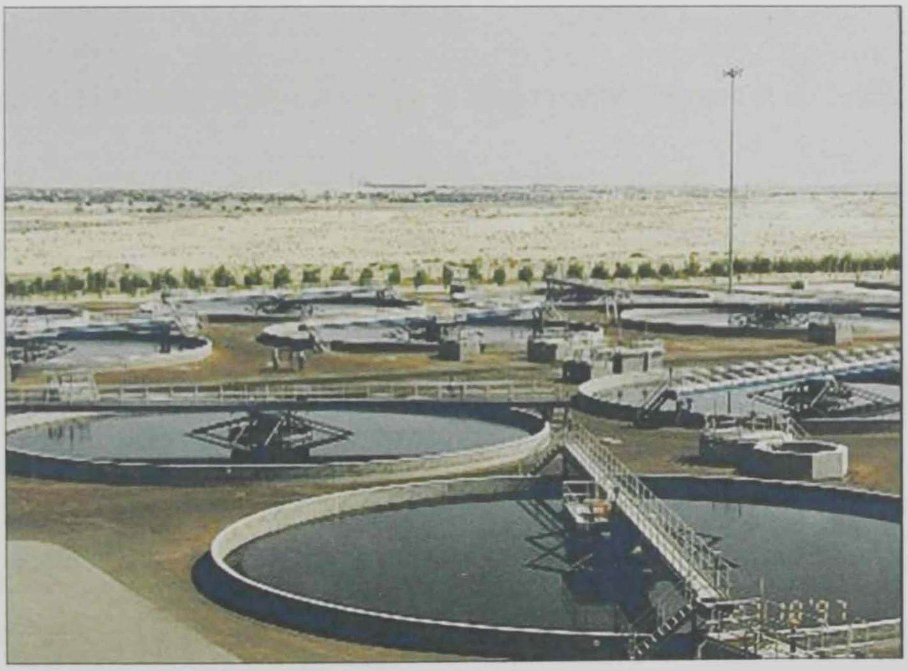

Photo 1.1 Mafraq waste treatment plant

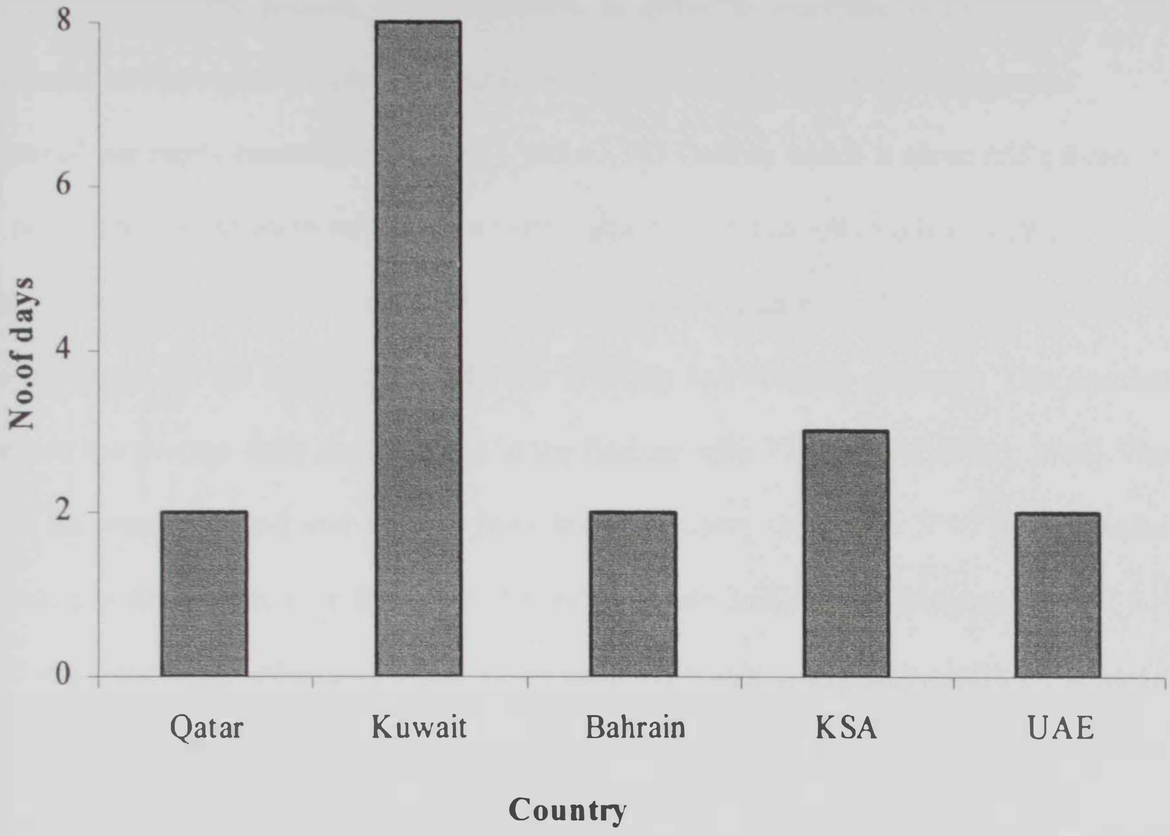

Figure 1.11 Number of days of storage of GCC countries [Almulla et al., 2005]

Table 1.6 Storage Capacity of ADWEA plants

Company	Storage Capacity (Mm^3)
Umm Al Nar Power Company	0.45
Bainounah Power Company	0.14
Al Mirfa Power Company	0.11
Emirates CMS Power Company	0.23
Gulf Total Tractable Power Company	0.32
Al Taweelah Power Company	0.23
Total	1.51

Water crisis management

Crisis management refers to conditions where a minimum water supply is required to be maintained during the periods of emergencies. In order to meet the water demands and maintain an uninterrupted supply, the Abu Dhabi Emirate needs a large storage system.

The annual per capita consumption in 2001 was 63,345 Gallons which is about 655 l/d/capita. This per capita consumption represents a very high rate as compared to other countries in the world. Under emergency conditions, this figure of consumption could be reduced by 0.45 % to be restricted for the use of freshwater for drinking and cooking purposes. This number represents the average daily domestic use in the Emirate with 77 g/c.d [ADWEC, 2004]. The rest of the water demand can be met from brackish water supply. In 2010 the estimated population in the emirate is in the order of 1,645,000. Assuming that the emergency will last for 90 days, the total volume of water to be supplied under emergency condition is about 11,400 MG as shown in Table 1.7. It should be emphasized that this is the minimum storage volume that should be available at all the time.

Table 1.7 Water demand requirement under emergency condition

Year	Per capita consumption (gallon/capita/d)	Minimum water required during crisis conditions (gallon/capita/d)	Population Million	Required volume of water	
				MG	Mm^3
2010	177	77	1.645	11,400	43.153

1.5 Objective of the Current Study

The ultimate objective of this study is to investigate the environmental impacts of groundwater storage in aquifers that might be contaminated from farms and/or due to leakage from existing landfills in the vicinity of groundwater protection zones. The specific objectives of the study include:

1. To identify the geometric, geologic, hydraulic and hydrogeological parameters of the Liwa aquifer system.
2. To simulate the current groundwater condition using a numerical model.
3. To define groundwater pattern under different recharge scenarios and define the protection zones.

1.6 Scope and Limitations

In order to achieve the objectives of the study, a comprehensive review of the study area and its surrounding including the hydrogeological conditions and aquifer properties was made. A numerical model was then implemented. Data was mostly available although some interpolation using Kriging techniques was needed. The study is devoted to the assessment of the environmental impacts of recharge in Liwa area. The limitations of the study include:

1. Lack of continues recorded data regarding the groundwater levels.
2. The reability of the calibration and the sensitivity of the model to various parameters; some assumption have been assigned for the natural recharge in Liwa area to match the results of the numerical model with the field results.
3. Reactions between the soil and the contaminants are not considered. Decay and retardation of contaminates are not included.
4. Results are more qualitative than quantitative. The time required to achieve specific information level may not be very much accurate.
5. A homogenous lithology is assumed in each layer and boundary conditions were assumed to represent the field conditions.
6. Abstraction of groundwater for irrigation purpose is not considered in this study. Anthropogenic effect is not included.

7. Evaporation from Sabkha area is based on estimated natural groundwater levels.

8. The high natural recharge in Western Region is estimated.

1.7 Organization of the Thesis

The United Arab Emirates has witnessed a remarkable development in many aspects of life over the past 35 years. This rapid development into a modern urban environment and the accompanying surge in population, caused by a major influx of foreign labour, have been imposing tremendous pressure on the natural resources including water.

Conventional water resources are limited in the UAE. Surface water is almost absent due to the scarcity of rainfall coupled with the arid conditions. Groundwater is mostly brackish and non-renewable. Over-pumping practices have resulted in a severe decline in groundwater levels and quality. A large portion of the freshwater demands in the UAE is provided by desalinated water which is expensive to produce.

This study is devoted to the qualitative assessment of the feasibility of artificial recharge in Liwa aquifer. The study examines the potential sources of contamination that may effect the study area. It includes a comprehensive review for various aspects related to groundwater resources in Liwa aquifer encompassing hydrogeology, hydrology and the aquifer properties. The thesis is composed of six chapters.

Chapter One presents an introduction for the Abu Dhabi Emirate. The physical setting of the Emirate and the prevailing climatic conditions are presented. The current water resources in the Emirate are quantified based on pervious investigation done by different researchers. These components include rainfall, groundwater, desalinated water as well as treated wastewater. The Objective of the study and the scope and limitation are also included at the end of this chapter.

Chapter Two is devoted to the background of groundwater recharge. A compressive review of the available methods for the groundwater recharge is presented. Artificial recharge systems including spreading methods and types that are the commonly used are explained in this chapter. Artificial recharge techniques have many benefits and several constraints which limit the possibility of conducing the artificial recharge systems. Appropriate condition for the artificial recharge is presented in this chapter. The selection of the appropriate methods to be used is also documented. The chapter is concluded with case studies of artificial recharge systems in the Arabian Gulf countries (Kuwait, UAE and Qatar).

Chapter Three presents a comprehensive review of the physical setting of the study area. The setting of Liwa area including the quality of native groundwater and the site selection criteria in this study area are presented. This criteria includes a review of some parameters such as the land use nearby the study area, the hydraulic properties, the existence of protection layer, the distance to desalination plants as a source for water recharge, infiltration capacity, etc. Aquifer properties and the possibility of evaporation loss or natural recharge and surface recharge are also presented. Finally the chapter is concluded with an overview of conducted field tests and laboratory analysis, completed within the activities of the Groundwater Assessment Project- GTZ, is presented.

Chapter Four is divided into two main parts; studying the feasibly of artificial recharge in the study area and determination of the protection zone of the well fields. The groundwater model FEFLOW (Finite Element Flow) is used in this study. An introduction to the methodology that has been used is present in the first section of the chapter including the preparation of the conceptual model of the study domain. The steps of preparing the model and georeferenced data using Arcmap and FEFLOW are elaborated. The mesh design with a specified number of elements and nodes is explained. Starting the simulation with 2D as an essential step to test the functionality of the numerical model, the 3D model is then explained. The boundary conditions and the hydraulic parameters are also discussed in this chapter. Steady state modelling is presented in a separate section with a focus on the calibration and verification of the model. The two scenarios of using the hydraulic head distribution and pumping test results are elaborated. The groundwater balance resulted from the model is presented at the end of this chapter.

Chapter Five is devoted to the results and the discussions. The chapter is divided into two parts. The first part presents the results and the feasibly of artificial recharge in the study area and the second part is concerned with the determination of the protection zone of the well fields. The first part includes all the information related to the artificial recharge. Arrangement of wells for recharge and recovery process, number of wells and rates of water injection are included. The rise of groundwater levels as a result of injection of desalinated water is presented in maps for the several durations of different injection rates. The second part is devoted to the environmental impacts and determination of the protection zone around the well fields. Potential contamination sources were identified and the well head protection zone is studied through different methods including the Arbitrary Fixed Radius (AFR), Calculated Fixed Radius (CFR) and time of travel (TOT) using FEFLOW.

Chapter six outlines the conclusions and the recommendations of the study.

A list of references that were used in the study and two appendices are attached.

Chapter II

Groundwater Recharge

2 Groundwater Recharge

Groundwater recharge could be encountered by natural recharge, enhanced recharge, incidental recharge and artificial recharge.

Natural recharge is the difference between water inputs into the soil (precipitations and infiltration from stream, or other natural water bodies) and water outputs (evapotranspiration plus runoff). Table 2.1 shows the different percentage of natural recharge from precipitation with respect to climate conditions.

Table 2.1 Natural recharges as percentage from precipitations

Climate	Natural recharge as percentage from precipitation (%)
Temperate humid	30-50
Mediterranean	10-20
Dry	0-2

Enhanced recharge is replacing deep rooted vegetation by shallow rooted vegetation, or by changing to plants that intercept less precipitation with their foliage, thus increasing the amount of water that reaches the soil [Rome, 1993 and Bouwer, 2002].

Incidental recharge is defined as any human activities that are not intended for recharge of groundwater. Examples of these activities are sewage disposal by septic tank and or drainage or deep percolation from irrigated fields. Urbanization is another form of the incidental recharge, where most of the land is covered with streets and other impermeable surfaces that produce more runoff and have much less evapotranspiration than natural surfaces.

Artificial recharge systems are engineered systems where water is put on or in the ground for infiltration and subsequent movement to aquifer to augment groundwater resources. Artificial recharge is a technology for storing large water volumes in aquifers during times when water is available and recovering this water when needed to meet peak emergency, long-term storage or other needs [Gale et al., 2002 and Radaidh, 1993]. Artificial recharge could also be done through spreading water on the surface to increase the water table gradient from a source of recharge [Sadek and El Fadel, 1998 and Pallas, 1993]. Artificial groundwater recharge is already carried out in many countries all over the world and its applications were implemented about 50 years back while a few related applications were practised more than 100 years back. Most artificial groundwater recharge is still limited to surface application such as basins. Recently an increasing amount of recharge is carried out through wells.

2.1 Artificial Recharge of Groundwater

Artificial recharge is one of the many techniques used to manage water resources and should be used in conjunction with a wide range of others, including surface storage and demand management. Groundwater recharge is preferred because there are negligible evaporation losses, the water is not vulnerable to secondary contamination by animals or humans, and there are no algae blooms resulting in decreasing surface quality [Gale et al., 2002 and Bouwer, 2002]. Many objectives or advantages exist for this technique and will be described in the next sections.

2.1.1 Artificial recharge systems

Artificial recharge can occur through direct and indirect processes. The use or purpose of recharge like the maximization of storage (including seasonal, long-term, and drought or emergency water supplies), physical management of the aquifer, water quality management, management of water distribution systems, and ecological benefits may determine what method might be implemented for artificial recharge [NEIWPC, 2004 and Huisman and Olsthoorm, 1992].

Direct methods can be divided into surface recharge techniques and subsurface recharge techniques. In surface recharge, water moves from the land surface to the aquifer by means of infiltration through the soil. The surface is usually excavated and water is added to spreading basins, ditches, pits, shafts, perennial dams and allowed to infiltrate. Direct surface methods involve low construction costs and are easy to operate and maintain, in comparison to other methods [NEIWPC, 2004]. Indirect methods include modifying aquifers to enhance groundwater reserves. Numerous schemes exist to artificially recharge groundwater systems. Broadly; they can be grouped into the following categories:

+ **Spreading methods**
  - Infiltration or recharge basins
  - Perennial dams
  - Ditches and furrow
  - Irrigation

+**Drilled well**

Many schemes require low levels of technology and can be implemented with little engineering knowledge. Perennial dams require more engineering design and knowledge, increasing further when using drilled wells and boreholes for injection or for Aquifer Storage Recovery. Although simple in principle the efficient operation of spreading basins and infiltration schemes needs a good knowledge of the physical, hydraulic, geochemical and microbiological processes in operation and how to mange them for optimum performance. The description of each method is given in the following sections.

2.1.1.1 Spreading methods

Water spreading is applied in cases where the aquifer to be recharged is at or near to the ground surface. Recharge is achieved by infiltration through permeable material at the surface, which is managed to maintain infiltration rates. In situations where there is a reliable source of good-quality input water, and spreading infiltration can be operated throughout the year, then hydraulic loadings of typically 30 m/yr can be achieved for fine texture soils like sandy loams, 100 m/yr for loamy soils and 300 m/yr for medium clean sands and 500 m/yr for coarse clean sands [Bouwer, 2002]. Evaporation rates from open water surfaces range from about 0.4 m/yr for cool wet climates to 2.4 m/yr for warm dry climates. Thus evaporation losses are quit small as compared to hydraulic loading rates [Gale et al., 2002].

This is the most common, cost -effective and widely accepted method for aquifer recharge [Pallas, 1993 and Viswanathan and AlSenafy, 1998].

Infiltration or recharge basins

An infiltration basin is either excavated in the ground or it comprises of an area of land surrounded by a bank, which retains the recharge water until it has infiltrated through the base of the basin. If the underlying aquifer is reasonably permeable, a simple dug basin can be used. If the aquifer material is fine, rapid clogging will occur. In this case, covering the bottom and sides with an approximately 0.5 m thick layer of medium sand can retard the clogging process and extend the recharge periods in the facility.

The depth of the basin should be shallow enough, to allow rapid draining in cases where cleaning of the basin by drying and scraping is necessary. On the other hand, depths should be large enough to prevent deep penetration of sun light, which would result in rooted aquatic growth and consequent resistance to the lateral flow of water. Large areas of land have to be made available for infiltration basins and the method delivers a relatively low recharge rate per unit area utilised [Pallas, 1993 and Viswanathan and AlSenafy, 1998]. Aquifers should be unconfined and sufficiently trasnmissive to accommodate lateral flow of the infiltrated water away from the recharge area without forming high groundwater mounds that interfere with the infiltration process. Clogging of the basin floor is the predominant problem during recharge, creating a filter skin on the bottom and sides of the spreading basin [Bouwer, 2000 and NEIWPC, 2004]. Hence, water sources for this system should be of adequate quality to prevent undue clogging of the infiltration surface by deposition and accumulation of infiltration surface by [Bouwer, 2002 and Perez and Carrera, 1998]:

1. Deposition and accumulation of suspended solids (sediments, algae and sludge)
2. Formation of bio films and biomass on and in the soil.

3. Precipitation of calcium carbonate or other salts on and in the soil.
4. Formulation of gases that stay entrapped in the soil, where they block pores and reduces hydraulic conductivity.

To counteract the above problems, the following methods should be considered:

1. Raise the water level in the basin to increase the driving head.
2. Apply a rotational system of water spreading and drying and subsequent scraping of the basin. Drying kills microbial growth, and this, combined with scraping of the basin bottom, reopens soil pores.
3. Mechanical treatment of the recharge water should be done if needed by primary sedimentation to remove suspended solids. Settling efficiency can be increased by addition of flocculating chemicals.
4. Chlorination of the recharge water to prevent microbial activity.
5. Mechanical treatment of the soil by ploughing to increase permeability.
6. Lining the basin with a layer of medium sand to act as a filter to remove suspended solids.
7. Pre-treatment of water to reduce suspended solids, nutrients and organic carbon.

Perennial dams

Semi-perennial or perennial dams gather larger quantities and depths of water, which can be used both as a source of water for direct irrigation as well as for increasing groundwater recharge. Generally, good dam sites are becoming scare and a silting over successive periods of inflow will lead to a reduction in the effectiveness of the recharge structure. The hydraulic head resulting from the accumulation of several meters of water will also force fines further into the infiltration surface and will compact the sediments, further reducing their effectiveness. All these factors should be appreciated and, if possible, quantified to ensure head resulting from the accumulation of several meters of water will also force fines further into the infiltration surface and will compact the sediments, further reducing their effectiveness. All these factors should be appreciated and, if possible, quantified to ensure that the dams are managed as either recharge or storage structures. Abu Dhabi Emirate topography is generally not suitable for the construction of recharge dams [EAD, 2005]. In fact, only one recharge structure, a diversion dam with several downstream recharge basins, exists in the Emirate at Al Shwaib. It diverts Wadi Sumeni into a series of recharge basins as it enters Abu Dhabi territory and has a combined storage capacity of 31.5 Mm^3. The main beneficiary of the enhanced recharge is agriculture. Numerous farms exist immediately south of the diversion structure. Figure 2.1 shows the main channel in Shwaib. Dams have various disadvantages such as evaporation losses, sediment accumulation, potential of structural failure and adverse ecological and environmental impacts.

Figure 2.1 Shwaib – wadi diversion structure - main channel

the root zone, the intention is not to recharge the aquifers. The net outcome may be recharge with poor quality water leading to rising groundwater levels and water logging. Water can also be spread deliberately during dormant or non -irrigating seasons specifically for artificial recharge. As a distribution system is already in place, no additional cost for land preparation is needed. Irrigation is, however, often carried out on flat plains where the water table is at shallow depths. Even where the groundwater level is originally found at a considerable depth, deep percolation of water applied to leach salts from the root zone, can result in a water table rise to a shallow depth. The quality of water under irrigated areas also needs to be assessed carefully as it may contain unacceptable concentrations of leached salt as well as residual agricultural chemicals [Gale et al., 2002].

Ditches and furrow

A ditch is a type of recharge and it is a long narrow trench with its bottom width less that its depth. Clogging is the main cause of declining recharge rates. Elimination of the clogging of the ditches could be achieved as explained earlier in this chapter. Such systems can be easily designed to fit the topographic and geologic conditions at a site.

2.1.1.2 Drilled wells and boreholes

Well or borehole recharge is used where low permeability strata overlie target aquifers, in order to recharge water directly into the aquifer. This technique is suitable for deep-seated aquifers and has the advantage that the recharge water can bypass thick impervious layers to be introduced to the most permeable portions of the aquifer. Recharge wells are also advantageous when land is scarce. However, recharge water quality requirements are usually significantly higher for borehole injection than for groundwater recharge by means of spreading [Gale et al., 2002 and Viswanathan and AlSenafy, 1998]. Aquifer Storage and Recovery (ASR) is the more popular technique to store water in aquifers during times when water is available, and recover it back from the same injection well during times when it is needed. More than 50 ASR facilities exist in United State using ASR technology [NEIWPC, 2004]. Clogging of aquifer material or the borehole screen either by suspended sediment, entrained air in the recharge water, microbial growth or chemical precipitation due to incompatibility of native and recharge water is a common problem encountered, leading to excessive build-up of water levels in the recharge well. These clogging processes of the wells can be managed by:

1. Mechanical treatment of the recharge water by plain sedimentation or filtration to remove suspended solids.
2. Pre-treatment of water to remove suspended solids.
3. Settling efficiency can be increased by addition of flocculating chemicals. Additionally, the water should not be allowed to cascade into the borehole, as this will entrain air that will clog the aquifer. Water should be introduced through a valve to ensure a continuous column to the surface. Some chemical pre-treatment of the water may be required to prevent flocculation of iron, $CaCO_3$, etc. and chlorination or other disinfection may be needed to prevent microbial growth.
4. Clogged wells may need to be recovered at regular intervals using surging and pumping to remove fines and bacterial growth physically and the use of a wetting agent to remove air in an air-clogged well. This method is the most efficient method.

2.1.2 Benefits and constraints

The purpose and advantages of artificial groundwater recharge include:

1. Replenishment of depleted aquifer systems.
2. To meet the variation in the seasonal demand. By storing water underground and recovering it later using ASR techniques, it is possible to develop water when needed,

especially during the drier months or as per demand [Gale et al., 2002, Viswanathan and AlSenafy, 1998, NEIWPC, 2004 and Bouwer, 2002].

3. Creation of a drinking water supply back-up system in case of emergency [Viswanathan and AlSenafy, 1998 and Bouwer, 2002].
4. Creation of strategic reserves. Surface engineering constructions of similar sizes are very expensive if not impossible to build.
5. The quality of the native groundwater can be improved by recharging with high quality injected water through soil aquifer treatment [NEIWPC, 2004 and Bouwer, 2002].
6. In aquifer systems, the artificially created reserve is safe in terms of hygiene.
7. Depending on the applied technology, evaporation or seepage losses can be eliminated [Bouwer, 2002, David and Pyne, 1995].
8. Artificial recharge can significantly increase the sustainable yield of an aquifer and is environmentally attractive, particularly in arid regions [David, 1998, David and Pyne, 1995].
9. The technology is appropriate and generally well understood by both the technicians and the general population [David, 1998 and David and Pyne, 1995].
10. Very low cost relative to other water supply and treatment alternatives [David and Pyne, 1995].
11. Aquifer recharge systems are easy to operate if well designed [David, 1998 and Abu-Taleb, 1993].
12. ASR-surface structures require minimal land as compared to other techniques [Bouwer, 2002, David and Pyne, 1995].
13. Reduce seawater intrusion and land subsidence [Bouwer, 2002].

14. Relatively simple technology to design, construct and operates [David and Pyne, 1995].

15. Proven performance in the many countries, including the United State, Holland and England.

There are some limitations or constraints in implementation of recharge and recovery techniques which slow down or discourage the implementation. Some of the limitations are listed here:

1. In the absence of financial incentives, laws, or other regulations to encourage landowners to adequately maintain wells. The wells may ultimately become sources of groundwater contamination [NEIWPC, 2004].

2. There is a potential for contamination of the groundwater from injected surface water runoff, especially from agricultural fields and roads. In most cases, the surface water runoff is not pre-treated before injection.

3. Recharge can degrade the aquifer unless the quality control of the injected water is adequate.

4. Unless significant volumes can be injected into an aquifer, groundwater recharge may not be economically feasible.

5. The hydrogeology of an aquifer should be investigated and understood before any full-scale recharge projects are implemented. In karstic terrain, dye tracer studies can assist in acquiring this knowledge.

6. During the construction of water traps, disturbances of soil and vegetation cover may cause environmental damage to the project area.

7. Lack of local experience in ASR.

8. Difficulty in finding suitable hydrologic conditions suitable for water storage [NEIWPC, 2004].

2.1.3 Appropriate conditions for artificial recharge

The effectiveness of artificial recharge schemes are governed by climate, geology and hydrogeology, site selection, topography, source water availability and quality, artificial recharge methods and potential costs and benefits and environmental consideration [Gale et al.,2002]. The complex interaction of some or all of these factors determines the degree of success, which can be viewed from a variety of perspective. Table 2.2 gives an initial assessment on whether the site can be considered for the implementation of artificial recharge projects or not. This table shows general guidelines for the selection of artificial recharge sites. For example, unconfined aquifer with small porosity and transmisivity will be of low receptivity to artificial recharge [Rimawi et al., 1993]. The table gives a first indication about the suitability of the aquifer for recharge. These factors will be discussed for Liwa area. The proper selection of the site is very important to ensure the success of any artificial recharge projects. The selection criteria should be based on the following:

1. Geological set-up of the location (e.g. type of aquifer, lithology, lateral and vertical extension, homogeneity [Rimawi et al., 1993 and Pallas,1993])

2. Geohydraulic aquifer properties (e.g. transmisivity of the aquifer should be moderate (150-400 m^2/d) [Mukhopadhyay et al., 1998]) to allow water to move rapidly from the mound created under the recharge basin but should not be too high so that water cannot be recovered.

3. Depth to water level should not be less than 5 to 10 m [Rome,1993 and Pallas,1993]

4. Infrastructural aspects (e.g. proposed site should be near to the injection water source [Mukhopadhyay et al., 1998].

5. Surface material has to be highly permeable to allow water to percolate easily.

6. The unsaturated zone should present a high vertical permeability, and vertical flow of

water should not be retrained by less permeable clay layers [Rome, 1993, واتق رسول اغا 1993 and Pallas, 1993].

7. Environmental consideration and availability of data [David and Pyne,1995]

8. Quality of the native groundwater (e.g. salinity of the groundwater should not exceed 5,000 mg/l [Mukhopadhyay et al., 1998]).

9. Quality of the resulting mixed water (reactions with native groundwater and aquifer materials) is also an important factor to be taken into account.

2.1.4 General criteria

The physical success of an artificial recharge scheme depends largely on the local hydrogeological conditions. These determine the ability of the water to percolate through the unsaturated zone and the ability of the aquifer to store the recharged water. The main factors to consider include:

1. Hydrogeological properties of the aquifer and the overlying formations.
2. Physical and hydraulic boundaries of the aquifer.
3. Depth to aquifer

The hydrogeological conditions in the surface and unsaturated zones are most important for schemes using spreading techniques, as water must move downward through these zones before reaching the aquifer. The percolation rate depends on the vertical permeability of the unsaturated zone. Once the recharge water reaches the water table, the amount of water the aquifer is able to store depends on its hydraulic characteristics (transmisivity, porosity, etc.) and its thickness and aerial extent. The receiving formation must have sufficient permeability and thickness to accept recharged water at a designated rate. On the other hand, aquifers with high hydraulic conductivities can result in rapid dispersal of the recharge water and, as a result, only limited quantities of water can be recovered.

The best water depth from the stand point of maximizing infiltration rates and minimizing adverse effects in the basin is determined by the local conditions and is best evaluated by on site experimentation [ASCE, 2001]. High hydraulic head in a surface recharge system can produce higher infiltration rate but may also tend to compress clogging layer once they form. The depth to water table will determine which artificial recharge methodology is most applicable for the site. When an aquifer is located at, or near to the ground surface, water spreading may be applied, e.g. by recharge basins. When a thin layer of less permeable material covers an aquifer, this layer may be ripped open by ploughing or harrowing after which the same spreading methods can be used. If however, the less permeable layer is thicker, recharge may only be applied via pits or trenches if they can be made deep enough to penetrate the confining layer. Where the low permeability layer is of greater thickness, or the receiving aquifer is located at depth, recharge can only be accomplished using wells or boreholes. In addition to these subsurface characteristics, a hydrological and environmental assessment should also consider the impacts of these schemes in the area including the impacts on existing users of groundwater and the proximity of potential sources of contamination.

Table 2.2 Identification of the important parameters used in groundwater artificial recharge systems classification [Rimawi et al., 1993]

1-System Description

Aquifer	**Condition**
1- Unconfined	(A) Dominate aquifer
	(B) Minor or not very productive
	(C) Hydrological insignificant
2- Confined	(A) Dominant aquifer (thick)
	(B) Multiple thin and productive
	(C) Not highly productive
	(D) Hydrological insignificant

Aquifer Type	**Combination of conditions**				
1- Unconfined	(A)	(A)	(B)/(C)	(B)	(C)
		With			
2- Confined	(A)/(B)	(C)/(D)	(A)/(B)	(B)/(C)	(C)/(D)
Combination Value:	1	2	3	4	5
	Most favourable			Least favourable	

2- Storage and Transmission Characteristics

Porosity	(L) Large (0.2)
	(M) Moderate (0.1-0.2)
	(S) Small (0.01-0.2)
Transmissivity	(L) Large (2500 m^2/day)
	(M) Moderate (250-2500 m^2/day)
	(S) Small (25-250 m^2/day)
	(V) Very Small (25 m^2/day)

Receptivity Determination

Variable	**High**	**Moderate**	**Low**
System	1, 2 , 3	1, 2, 3, 4, 5	3, 4, 5
		with	
Storage and transmission	LL, ML	LM, LS, LV, MM, SL, SM	MS, MV, SS, SV

2.1.5 Topography

Topography is an important factor to consider with regard to recharge site selection. It permits or retards runoff, thus influencing recharged water infiltration rates and amounts. Specific attention is needed in the case of recharge to unconfined aquifers, where the water table may rise during recharge and may reach the ground surface. Groundwater levels tend to form a surface that is sub-parallel to the topography. Areas of low topographic relief therefore tend to have very low groundwater gradients as compared to areas with higher relief. This factor should be considered in relation to movement of water away from the location where it is recharged artificially. The depth to groundwater is also generally related to topography, the water table being shallower in areas of low relief and hence less recharge capacity is expected in such areas.

2.1.6 Source water consideration

A prerequisite for artificial recharge of groundwater is the availability of a source of water of suitable quality, in a sufficient quantity. Several sources of water can be considered for use as recharge water, namely groundwater, potable and desalinated water and treated wastewater.

2.1.6.1 Groundwater

In areas where potable groundwater is only available on a seasonal basis, it can be used as a source of recharge water to resolve seasonal supply and demand imbalances. In these special circumstances, groundwater may be abstracted from a low-yielding aquifer and stored in an adjacent or overlying aquifer. This water is then available at the point of demand in periods when the use of groundwater is restricted, or to meet peak demands, e.g. during summer.

Where groundwater of non-potable quality is used as the recharge source, the usable quantity of water recovered is affected by the degree of mixing with native groundwater during storage. Increasing the time between recharge and withdrawal increases the mixing between the introduced and native groundwater. Untreated groundwater is frequently of excellent quality. However, when recovered, the stored groundwater often requires treatment to meet potable standards. Where recovery efficiency is acceptable, it is more cost-effective to store and recover treated drinking water rather than untreated groundwater as no water treatment is required after recovery.

2.1.6.2 Potable and desalinated water

Potable and desalinated water is a major source of recharge water used in Aquifer Storage/Recovery schemes. High-quality treated water is injected through wells, in the case of confined aquifers and through recharge basin in unconfined aquifers, to create a bubble of potable water in the aquifer. These bubbles can be created in non-potable aquifers by displacing the native water and have proved to be a cost-effective and environmentally sustainable method for resolving a wide variety of problems. The schemes are usually constructed near treatment works or desalination plants, the source of the recharge water, to save water transportation cost and to utilize the surplus of treatment facilities.

In arid areas, such as the Arabian Gulf region, where water demand exceeds the available water from renewable resources, freshwater from desalination plants is used to bridge this gap. To ensure water availability during emergencies, for example, when desalination plants are out of commission, large freshwater storage capacities are required. Field trials have been undertaken to evaluate the feasibility of introducing desalinated water into aquifers to build up this freshwater reservoir.

Due to the high quality of the desalinated water, no major geochemical compatibility problems are expected as the water can be treated to minimize any potential reactions with the aquifer material; for example the pH can be adjusted to be non-aggressive.

2.1.6.3 Treated water

Various sources of water are available for groundwater recharge. In recent years, the use of non-conventional water resources such as recycled municipal wastewater, has received an increasing attention. The primary reasons for considering use of recycled water in groundwater recharge are that recycled wastewater is available for reuse at a relatively low cost. It also provides a dependable source of water even in drought years.

Wastewater as a source is of predictable volume with a fairly uniform rate of flow over time and of constant, but inferior quality. Wastewater requires significant treatment before being considered to be of acceptable quality for aquifer recharge and to minimize the extent of any degradation for groundwater quality. Variation in summer and winter demands can lead to have more than half of the water as surplus in winter seasons [Rashid, 2005]. Some countries dump this excess wastewater into the sea while it could be beneficially stored in the underground for later use.

Groundwater recharge with reclaimed wastewater can be regarded as a feasible approach to water reuse that results in the planned augmentation of groundwater for various beneficial uses. It can be used to lessen the long term supply versus demand imbalance encountered in the Emirate. The water levels in brackish groundwater fields are declining due to the over pumping to meet landscaping and gardening requirements. Recharging could help replenishment of such aquifers. Advantages of such techniques could increase the time of recycling and thereby allows more time for biodegradation of contaminations that degrade more slowly [Dillon et al., 2006]. Economics of water reuse might be considered the most important factor in determining the potential of water reuse. Such economic assessments could be influenced by several factors such as: (1) level of treatment, (2) geographical locations of users and (3) size of treatment plants [Hamoda, 2004].

Normally, treatment to drinking water standard is required if the recharge groundwater has a potential of being reused as potable supply [Viswanathan and AlSenafy, 1998]. Reclaimed wastewater can also be used for irrigation purpose. The other problem is psyological barrier that prevents people from using wastewater. Public acceptances, as well as the associated cost for pipelines, pumping stations, etc. to convey the water from wastewater treatment plants to where it is needed add constraints on the utilization of reclaimed wastewater. Use of the reclaimed wastewater for irrigation of fodder crops is more easily accepted than irrigating crops for direct human consumption.

A major consideration is the possible presence of chemical and microbiological agents in the source water that could be hazardous to human health and to the environment. Concerns about hazardous agents in the water apply particularly to potable reuse. Although nonpotable uses, such as irrigation, can result in human exposure to hazardous agents, there is less potential for exposure and the risks are therefore significantly lower.

The two possible constraints that may limit the use of recycled municipal wastewater for groundwater recharges are the impacts of emerging contaminants on long-term human health, and public perception on potable reuse. Minimizing the health risks caused by water is of great importance worldwide.

A further concern is how risks resulting from aquifer recharge will be defined accurately in terms of environmental and health issues (e.g. preventing the degradation or impairment of water quality in groundwater basins that are, or could be, used for domestic water supplies). Detailed information on the processes governing the fate of pathogens and chemicals is required in order to develop appropriate models for determining risk assessment. Human pathogens and trace organic compounds are of particular concern when groundwater recharge involves aquifers supplying domestic water.

2.1.7 Selection of the appropriate artificial recharge methods

There are many methods that are used widely in recharging aquifers. These methods differ from one site to another and also according to the source of recharged water, type of aquifer (confined/unconfined) and the hydrogeological parameter.

In addition to the characteristics mentioned in Table 2.2, a full feasibility assessment should also consider the impacts of the scheme on the hydrological and ecological regime in the area including the impacts on existing users of groundwater, the contribution of base flow to streams and the proximity of potential sources of contamination. The principal criteria for selecting the appropriate methods depend upon many factors including the source of water (Figure 2.2). For example, if a reclaimed water supply is available, recharge wells and spreading methods may be considered. For the natural occurring surface and groundwater the options vary between direct and indirect methods. In each type different techniques could be used [Rimawi et al., 1993].

The hydrogeological situation, local and regional demand for additional water storage and the availability and quality of the native groundwater are also important. Spreading techniques can be applied in a wide variety of forms where the surface (or near surface) layer is permeable. These techniques are comparatively simple and of low cost both in construction and maintenance. Where surface layers are impermeable and water can potentially be stored in the underlying aquifers then wells or boreholes are required to access these aquifers. Table 2.3 gives the major characteristics of aquifer recharge methodologies.

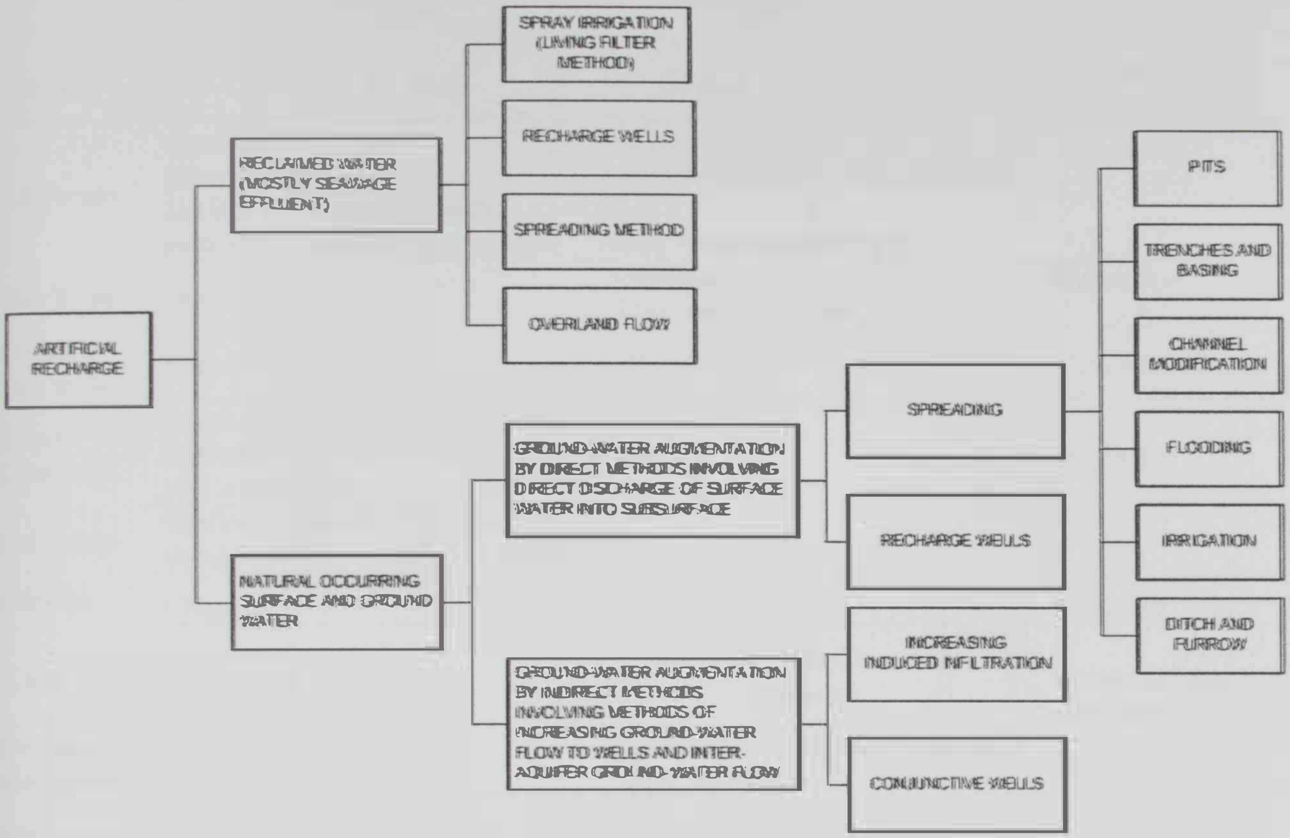

Figure 2.2 Principal methods of artificial recharge [Rimawi et al., 1993]

Table 2.3 Comparison of ASR methods [different sources]

Criteria	Method Recharge Basin	Drilled well or bore holes	Vadose zone injection well
ses- Process	Aquifer is at or near to groundwater Surface Infiltration through permeable material at the surface	1. Scarce of area 2.Recharge water can bypass thick layer to reach the permeable layer	
quifer Type	Unconfined	Confined Deep seated aquifer	Unconfined
Permeable urface soils	Required	Not required	Not required
Land equirements	Large area is required, and it is minimized if permeable surface soil is available	not large	not large
pth of water	Shallow; depth to water level should not be less than 5- 10 meter		
Problems	Clogging	Clogging	Clogging
Cost	Need large area	if used for injection/recovery, cost will be reduced	Relatively in-expensive
re-treatment equirements	Low technology	High Technology ,High quality of water is required	Removal of Solids
laintenance equirement	Drying and Scraping	Disinfection and flow Reversal	Drying and Disinfection
oil Aquifer Treatment	Vadose zone	Saturated zone	Vadose zone and Saturated zone
Capacity	1,000-20,000 m^3/ha-d	2,000-6,000 m^3/well-d	1,000-3,000 m^3/well-d

2.1.8 Applications of ASR in Arabian Gulf countries

There are several examples of using ASR in the world and in the Arabian Gulf. Kuwait, UAE and Qatar have conducted pilot projects of ASR [Gale et al., 2002 and Fox, 1999]. The anticipated water sources were excess desalinated water, flood water, and imported water. The water was either routed to recharge wells or infiltration ponds to build a strategic reserve to be used during water shortages. Early utilization of ASR on the Arabian Peninsula includes dams in mountainous areas of the eastern part and the collection of water from seasonal rainfalls to increase infiltration and storage for agriculture purposes [Fox, 1999]. The following section provides a summary of the implemented ASR project in selected GCC countries.

2.1.8.1 Kuwait

Excess desalinated water produced during the winter season in the past encouraged water authorities in Kuwait to evaluate artificial groundwater schemes. Artificial recharge of the Kuwait Group aquifer was first tried by Parsons (1964), using a recharge pit in the Rawdhatain depression for the purpose of collecting surface run-off during occasional rainstorms. Preliminary hydrogeological investigations indicated that the sandstone formations of Kuwait group at Rawadatain and the limestone Dammam formations at Sulabiya were potential recharge sites [Hutchinson, 1998]. The first recharge experiment consisted of infiltration tests in two recharge pits, and two well injection tests during the period 1961-1964. The objectives of the tests were to examine aquifer behavior in order to evaluate formation capability and potential clogging. Encouraging results were obtained from pits which showed high infiltration rates, leading subsequently to the construction of other pits in 1964. The well injection experiment at Rawadatain continued and in 1972 two injection tests were carried out, later followed by a 27 day long injection test in 1973. The long test provided results and indicated a water level rise from 8.2 to 15.2 m with reference to a specific datum. The decay of the water mound also monitored. Another injection test was made in 1977 to focus on water quality aspects.

Further efforts were made in the mid of 90's to investigate the other potential Dammam limestone formation at Sulabiya. Two sites were tested in Dammam formation, a limestone aquifer. One site was tested in the overlying Kuwait Group Aquifer, which is comprised of sand intervals between layers of cemented sandstone [David and Pyne, 1995]. Testing procedures included use of sodium fluorescein dye and tritium to assess the mixing between natural and injected water. A single injection-recovery cycle was utilized. Clogging problems resulting from suspended solids were encountered. Test results were not conclusive and suggested limited storage and recovery potential at the Sulabiya site [Hutchinson, 1998]. Both aquifers are brackish, with TDS levels ranging from 2,700 to 5,000 mg/l in the Dammam Formation and 3,900 mg/l in Kuwait Group Aquifer [Viswanathan and AlSenafy, 1998]. The highlights of these experimented are given in Table 2.4.

For Sulaibiya Well SU-10, a recharge volume of 4.3 MG was injected during a period of 30 days, at rates that ranges from 0.65 Megalitre/day (650 m^3/d) to 0.39 Megaliter/day (390 m^3/d). Background TDS in the aquifer was about 5,000 mg/L. Mixing characteristics were such that over 45% of the recharge volume is recovered before the TDS exceeded 2,000 mg/l in the recovered water, and 90% is recovered before the TDS concentration reached 3,000 mg/l. Test results indicated that this site would be suitable for a strategic water reserve [David and Pyne, 1995]. The results indicated that it is technically possible to inject water in the aquifers of Kuwait, and to recover a part of this water, mixed with native groundwater in different proportions [Viswanathan and AlSenafy, 1998].

2.1.8.2 United Arab Emirates

Storing excess water from desalination plants into aquifers for future recovery during times of need is a valuable option for the Emirates. Two pilot projects were done in the UAE. Before starting the pilot project, a simulation for groundwater recharge with desalinated water was done. The study was undertaken by National Drilling Company (NDC) in Al-Ain in 1998 where excess water from Umm Al-Nar and Taweelah plants was stored in the underground in the shallow aquifer system for future recovery. The aim of the study was to assess the feasibility of augmentation and revitalizing the critical groundwater resources of Al-Ain area. The result of the study indicates that the aquifer storage recovery is a viable alternative for augmenting the depleted aquifer [Hutchinson, 1998]. A computer model was developed to demonstrate such feasibility before conducting field tests. Three scenarios were tested as given in Table 2.5. The first simulation is a well model, in which 1,000 m^3/d of the water are injected for 200 consecutive days and then recovered at a rate of 1,000 m^3/d for 50 days on an 8-ASR-cycle schedule lasting 2,000 days. The simulated average dissolved-solids concentration of the recovered water is about 500 mg/l. The remaining simulated plume of freshwater with dissolved-solids concentration less than 1,500 mg/l occupied an area of 100,000 m2 and the total volume added to the aquifer storage was 1,200,000 m3.

The second simulation is a pond model, in which 1,000 m^3/d were infiltrated through the bottom of an open pond for 245 consecutive days and then a down gradient recovery well was pumped at 1,000 m^3/d for 120 days for a cyclic schedule lasting 1,825 days, or 5 years. This is a realistic recharge/recovery scheme from the standpoint that excess desalinated water may be available for storage during low demand periods and the water can be recovered during peak demand periods. The simulated plume of freshwater after 1,825 days, or five ASR cycles, occupied an area of 88,000 m^2 and a volume of 660,000 m^3 were added to the storage.

The first pilot project is located west of the highway from Madinat Zayed to Meziyrah and comprises an area of about 10 km in EW direction and about 1.5 km in NS direction. It is designed for an infiltration capacity of 500 m^3/h and a recovery capacity of 750 m^3/h. Shallow to medium deep aquifer north of the Liwa Crescent was selected in this study because of the existence of a large natural fresh groundwater lens north of the Liwa-Crescent (salinity less than 1,500 ppm, partly meeting the TDS-limit of the international World Health Organization drinking water standard (1,000 ppm), sufficient lateral extension and aquifer thickness, sufficient depth of groundwater table, relatively homogenous lithology, remoteness to already existing well fields and favourable hydrochemical conditions. The feasibility study has clearly shown that the recharge and efficient recovery of desalinated water into an existing freshwater aquifer is feasible on a large scale [GTZ-DCO/ADNOC, 2002].

The second pilot project was implement by the United Arab Emirates Mubadala Development Company (MDC) which plans to develop a 30 billion imperial gallon (BIG) reserve of fresh water by utilizing Aquifer Storage and Recovery (ASR) technology. The primary objective is strategic water storage for use during emergency periods. An additional benefit from the project will be to meet peak summer demands. The final ASR well field will operate at a capacity of 20 million imperial gallons per day (MIGD) with potential expansion to 100 MIGD. The water for the ASR system will be provided by the water desalination plant in Fujairah, transported to the Emirate of Abu Dhabi via the Fujairah water transmission pipeline. Identification and test a potential site for the ASR project implementation was done by defining the storage zone and confining formations, the aquifer thickness, and the related hydraulic parameters, and subsequently testing the identified aquifer's potential for ASR through pilot testing. Preliminary test results at the site indicated the site has potential for ASR development. Pilot testing of the ASR system at the selected site was then necessary to gain information needed to confirm the feasibility of implementing a large-scale ASR project.

The implementation of the second phase of the project necessitates the construction of two ASR wells for injection and production and was surrounded by twenty additional wells positioned to assess the evolution of water plume during injection and production. The implementation of the agreed cycles started in February 2006. It consisted of performing two cycles of Injection-storage-Production in one well to be followed by one injection cycle in two wells simultaneously that ended in December 2006. The two objectives of this phase were achieved and exceeded, the maximum production rate reached 500 KIG (Objective set at 450 KIG) and the final System Efficiency was in excess of 88% (Objective set at 85%).

2.1.8.3 Qatar

The groundwater system in Qatar is heavily over exploited mainly for irrigation purposes. The over abstraction has resulted to a significant deterioration in the water quality due to saline water intrusion from the coast and upconing of more saline water from deeper aquifers. Abstraction is currently almost more than five times the natural recharge of the aquifers [Esa, 1998]. Unless a remedial action is taken, the remaining fresh groundwater reserves of about (1500 Mm^3) will be exhausted, within a time scale of probably 10 to 12 years.

Artificial groundwater projects using runoff collection depressions and recharge wells in Qatar were initially implemented in 1977 through the use of five recharge wells located in depression areas. The system was eventually expanded to include 140 recharge wells. During the period 1977-1988, monitoring of the groundwater levels indicated that recharge volume had increased by 30%. Water level fluctuations indicated the response of wells to rainfall unoff events. Eight hundred additional recharge wells were planned for construction effective of 1994.

Past artificial groundwater activities in Qatar consisted of the use of a large scale flood water recharge well scheme implemented over most of the area of Qatar. More recently, a pilot recharge project has been initiated which involved large depressions where runoff usually collects and is then diverted to a number of recharging wells. The wells recharge the carbonate Rus and Umm er-Radhuma formations.

A feasibility study for large scale artificial recharge schemes was conducted to investigate the capability of the water bearing formations to store injected water, and to determine the efficiency of recovery for later use. The study was implemented on Rus and Umm er-Radhuma formations located in the northern region of the country, over a two year period from 1992-1994. The results of this large scale artificial recharge study identified the layers in both aquifers as having potential for building up groundwater reserves. Many artificial recharge scenarios were studied including the options of recharging the aquifers from either desalinated water or imported surface water for the purposes of building strategic reserves, enhancing capacity of existing well fields, supporting existing farms and control of saltwater intrusion. The results of the study identified the best areas for water injection and the optimum methodology for water injection and recovery.

Table 2.4 Highlights of the injection-recovery test [Viswanathan and AlSenafy, 1998]

Recharge details	Injection wells		
	SU-10	C-105	SU-135 A
Aquifer injected	Damman Formation	Damman Formation	Kuwait Group
Transmissibility(m^2/d)	50	≥ 4,000	250
TDS of formation (mg/L)	3,500	2,700	4,000
TDS of Injected (mg/l)	310-410	2,700	4,000
Average injection rate(m^3/d)	590	5,900	655-820
Total number of injection days(d)	30	30	0.9
Total injection volume(m^3)	16,416	171,700	642
Volume of recovered water at TDS<1500 mg/l (m^3)	2,727	25,730	409
Recovery efficiency	15-20%	10-15%	65-75%

Table 2.5 Scenarios tested by the model for Al-Ain region [Hutchinson, 1998]

Scenario	Recharge (m^3/d)	Recharge duration (days)	Recovery (m^3/d)	Recovery duration (days)
Well	1,000	200; with 8 ASR cycles lasting 2,000 days	1,000	50
Pond	1,000	245; with 5 ASR cycles lasting 1,825 days	1,000	120
Pond	1,000	1,098	15,000 With 15 wells	10

Chapter III

Physical Setting Of The Study Area

3 Aquifer Storage and Recovery Pilot Test

Site selection of the study area is presented in this chapter with some details. Physical settings including the location, land use and other parameters were taking into consideration in addition to the hydrogeological setting with geology and the type of aquifers.

3.1 Location Selection Criteria

The selection of the location of the current project was based on the following:

1. Existence of a large natural fresh groundwater lens north of the Liwa-Crescent (salinity less than 1,500 ppm, partly meeting the TDS-limit of the international WHO-drinking water standard (1,000 ppm)).
2. Sufficient lateral extension and thickness of the aquifer.
3. Sufficient depth to the groundwater table.
4. Relatively homogenous lithology of the system.
5. Remoteness of the site to already existing well fields and possible contamination sources.
6. The mixture of the artificially recharged desalinated and the naturally stored groundwater will improve the existing quality of the groundwater [Abu-Taleb, 1993].

The following aspects were also quantified [Rimawi et al., 1993]

- How much rechargeable water is available, when and at what rate?
- What is the quality of the recharge water?
- How much underground storage space is available, and at what depth?
- How readily will the aquifer accept the recharge water and how readily the water can be recovered?

- How will the quality of water change after recharge; and how quickly will the aquifer plug due the chemical, physical or bacterial processes?

The answers to the above questions are summarized in Table 3.1

Table 3.1 Summary of site selection criteria for the artificial recharge project

Item		Description
Land use	Agriculture Farming Industry	Farming activities are away (about 10-15 km) from the selected area. No indication of industry
Hydraulic properties	Transmissivity	Ranges from 100- 3,000 m^2/d with average of 1067 m^2/d
	Porosity	25-40 % ; average of 35 %
	Conductivity	2- 106 m/d ; average of 27 m/d
Protection Layer	Yes	This includes the Vadose zone and upper layer with a thickness of about 45 m. The area is covered with sand and is protected from possible contaminate
Distance to DSW availability	Close	The selected area is close to the main pipeline running along the road Madinat Zayed and Meziyrah of desalination sea water (DSW)
Movement of injected water	Possible	Possible under high transmissivity and steep gradient. However, both the transmissivity and the gradient are moderate in the study area.
Hydraulic gradient	Low	The hydraulic gradient is relatively low. The lateral groundwater velocity is in range of 10 m/yr.
Infiltration Capacity	High	Average infiltration rate per m^2 is 0.615 (m^3/h)
Groundwater Quality	Good	The native groundwater is suitable for drinking (within UAE/WHO limit)
Aquifer	Type	Unconfined aquifer. Vadose zone is the zone in which desalinated water will be injected.
	Material	Mainly sand dune in Western Region.
	Utilization	No significant pumping near to the selected site. Groundwater pumping in the vicinity of the site is not expected to have any effects.
	Thickness of the vadose zone	Vadose zone thickness around 45 m.
	Thickness	Aquifer thickness is around 90 m. The thickness of the aquitard is 50 m below the aquifer.
Possibility of	Evaporation loss	Does not affect the water balance, sufficient depth is selected to eliminate or reduce the evaporation loss.
	Natural Recharge	Natural recharge in Liwa area is erratic both in time and space, and can be considered as low.
	Surface Recharge	Surface recharge through basins is possible.

3.2 Physical Setting of the Study Area

Many parameters were taken into consideration when selecting the site for the artificial recharge. These parameters are explained in the following sections.

3.2.1 Location of the site

The Artificial recharge project is located in the northern part of the Liwa Crescent between Madinat Zayed and Meziyrah, east of the highway (Figure 3.1). The proposed location of the study area was selected in the vicinity of an available water supply that is also close to the distribution systems where major pumping stations are located [David and Pyne, 1995]. The study area was selected along the road Madinat Zayed and Meziyrah and thus reducing the costs of transporting water.

3.2.2 Land use

Human activities such as farming, digging wells for domestic or industrial proposes may result in producing liquid waste that might penetrate into the aquifer and cause groundwater pollution. Agricultural development may also pollute groundwater systems. Therefore, the study area was selected away from any human and/or agricultural activities. The nearest farming area is about 10-15 km from the recharge site and far away from oil fields and petrol stations.

3.2.3 Meteorological information

The only natural recharge in the area is the rainfall. Figure 3.2 and 3.3 show the meteorological station located near to the study area and the precipitation distribution between February 2001 and February 2005. Tracing of some elements and using numerical modelling within the activities of the Groundwater Assessment project- GTZ concluded that the groundwater flow originates from the centre of the area and flow to the edges of the domain indicating the existence of natural recharge. The water penetrates from the top of the hill and spread out in the aquifer in all direction so natural flashing takes place.

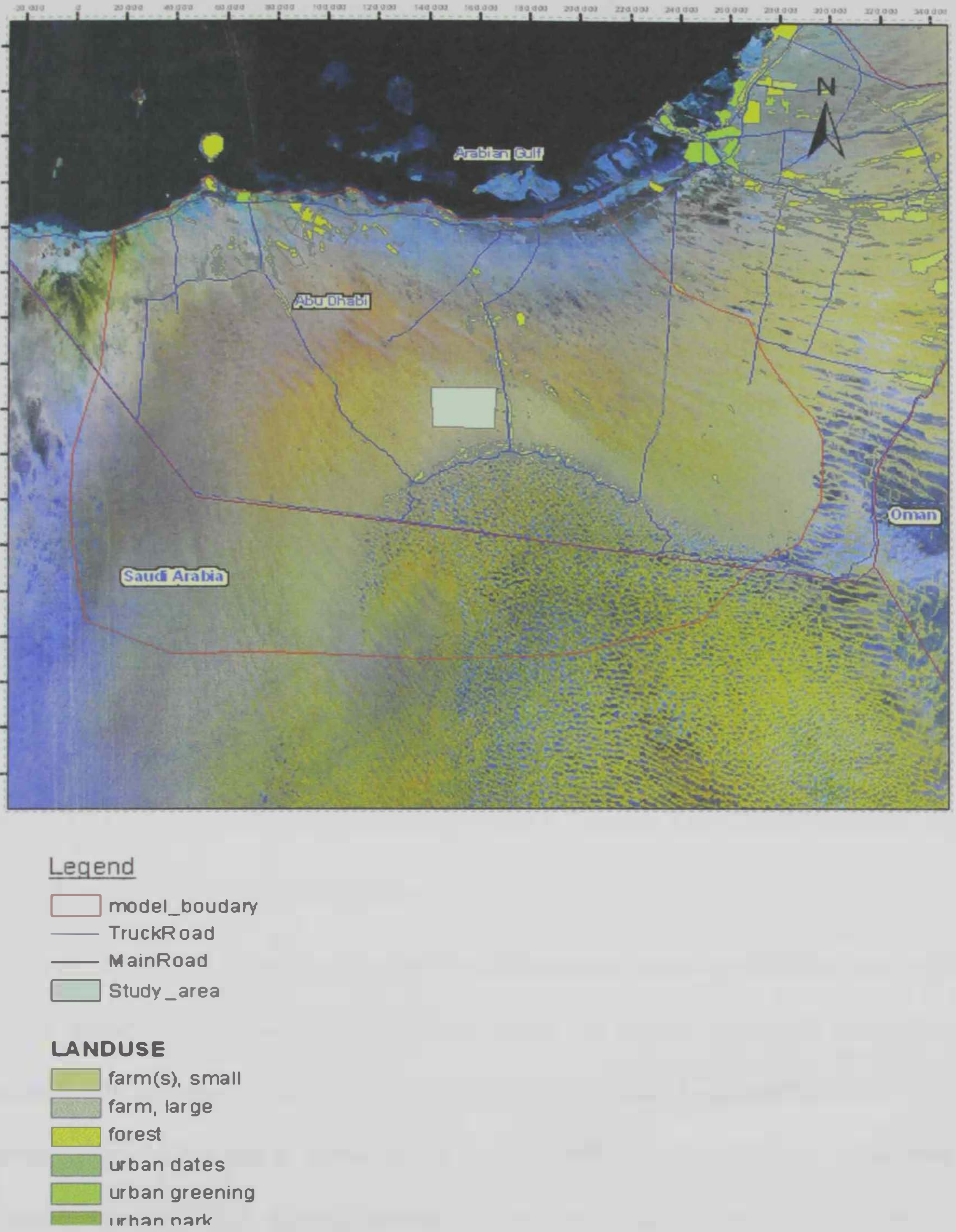

Figure 3.1 An overview of the study area

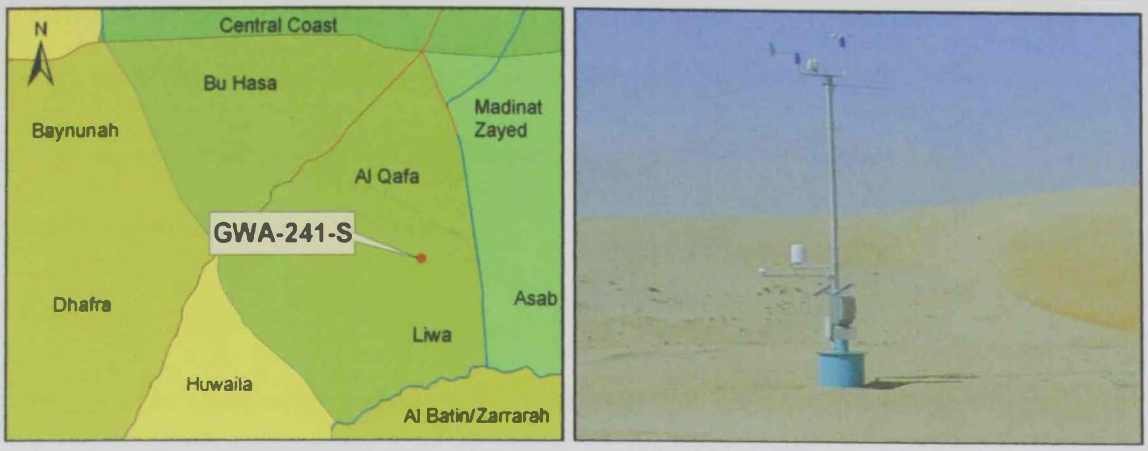

Figure 3.2 Hydrometeorological station, GWA-241-S [GTZ-DCO/ADNOC, 2003]

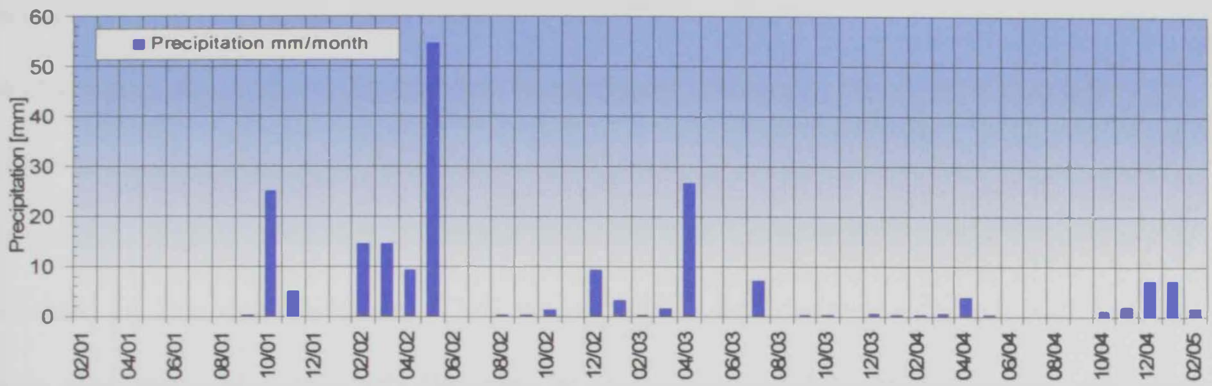

Figure 3.3 Precipitation [mm/month] from February 2001 to February 2005 [GTZ-DCO/ADNOC, 2003]

3.2.4 Water recourses availability

Continuous supply of water is an essential consideration before establishing any artificial recharge project. It is usually insufficient to know the average and peak rates of water availability for recharge, although this is required [David and Pyne, 1995]. The volume of recharge water for the whole period of recharge 16,286 MG (61.6Mm^3) is taken from the existing transmission main line of the desalination plant along the road from Madinat Zayed-Meziyrah. Tables 3.2 and 3.3 show the characteristics of desalinated water.

3.3 Hydrogeological Settings

The geology and hydrological setting of the study area are explained in the following section. Type of the aquifer with its extent, the hydraulic parameters and the quality of the native groundwater is also explained.

3.3.1 Geology

The lithology of the Liwa aquifer has two main stratigraphic units as follows:

Quaternary unit: Holocene and Pleistocene eolian fine to medium sands and interdunal deposits. The thickness of this unit varies between 100 m and 150 m, depending on the topographic height of the respective location within the study area.

The Quaternary unit may be divided into two subunits. The upper unit is characterized by the predominance of well –sorted, fairly loose eolian dune sands with occasional intercalations of fine-grained, slightly cemented interdunal deposits. The lower subunit of the Quaternary is dominated by interdunal deposits. They consist of caliche horizons with traces of organic matter, siltstones and even marls that may be interpreted as playa lake sediments and give evidence of more frequent pluvial periods in the Pleistocene.

Tertiary unit: mudstones, evaporates and clastics of Miocene age. This unit has a thickness of more than 350 m. The Tertiary unit can also be subdivided in an upper unit, consisting of mudstone layers and evaporates (gypsum, anhydrite, dolomite) of the Lower Fars Formation, and a lower subunit that is marked by the predominance of clastic sediments (sandstones, siltstones), and are intercalated with layers of mudstones and anhydrite. Table 3.4 summarizes the upper stratigraphic sequence in the study area. All wells are completed within the upper part of the shallow aquifer that consists mainly of a eolian fine sands with only slight variations in grain-size and colour. The samples that were taken from 26 selected boreholes at 10 ft intervals during drilling of wells indicated the existence of a very homogeneous sand body that can be considered almost ideal for the purpose of artificial groundwater storage and recovery.

Table 3.2 Chemical characteristics of desalinated water, organic compound [RSB, 2004]

Parameters	Units	Allowable Values
Trichloroethene	µg /l	30
Tetrachloromethene	µg /l	3
Tetrachloroethene	µg /l	10
Chloroform	µg /l	200
Polycyclic aromatic Hydrocarbons	µg /l	0.20
Trihalomethanes	µg /l	≤1
1,2 Dichloroethane	µg /l	3
Benzene	µg /l	1
Bromoform	µg /l	100
Dichloramethane	µg /l	5
Dibromochloromethane (DBCM))	µg /l	100
Bromodichloromeftvane (BDCM)	µg /l	60
Chlorobenzene	µg /l	300

Table 3.3 Chemical characteristics of desalinated water, inorganic compounds [RSB, 2004]

Parameters	Units	Allowable Values
Sulphate	mg/l	250
Magnesium	mg/l	30
Sodium	mg/l	150
Potassium	mg/l	12
Chloride	mg/l	250
Nitrate	mg/l	50
Nitrite	mg/l	0.10
Ammonium (ammonia and ammonium ions)	mg /l	0.50
Total Organic Carbon	mg /l	No increase
Dissolved or emulsified Hydrocarbons; mineral oil	mg/l	mg /1
Aluminum	mg/l	0.20
Iron	mg/l	0.20
Manganese	mg/l	0.05
Copper	mg/l	1
Zinc	mg/l	5
Phosphorus	mg/l	2.20
Fluoride	mg/l	1.50
Arsenic	µg/l	50
Cadmium	µg/l	5
Cyanides	µg/l	50
Chromium	µg/l	50
Mercury	µg/l	1
Nickel	µg/l	50
Lead	µg/l	10
Antimony	µg/l	5
Barium	µg/l	700
Boron	µg/l	1000

Table 3.4 Hydrogeologic framework of Western Abu Dhabi [GTZ-DCO/ADNOC, 2002]

Unit	Subunit	Description
Quarternary Unit		
Holocene+	Upper Subunit	eolian, loose fine to medium sand
Pleistocene	Lower Subunit	Interdunal deposits (caliche, silt, marl) and sand
Tertiary Unit	Upper Subunit	Mudstone and evaporates of the Lower Fars Formation
Miocene	Lower Subunit	Miocene clastics (sandstones, mudstones, anhydrite)

3.3.2 Hydrogeology

In the Western Region (Liwa, Dhafra, Baynunah and Jabal Barakah) the shallow aquifer formation consists of sand and sandstone (thickness of 13 m to more than 60 m) underlain by siltstone, claystone and evaporites. The groundwater level ranges from 60 m to 108 m amsl in the Liwa area. The shape of the groundwater table is convex, the convexity being oriented to the top, and elongated, striking in E-W direction. The summit is situated approximately 25 km north of Mizeiri'ah. The gradient of the groundwater table is mild in E-W direction (less than 0.5 m/km), moderate to the south (about 0.5 m/km), and steep to the north (more than 0.5 m/km) [GTZ-DCO/ADNOC, 2005]. The main aquifer of the Western Region consists of the upper subunit of the Quarternary sediments. It is situated in the Northern Liwa area. The aquifer extends to the Sabkha Matti area in the west and to the east it borders the gravel plains in front of the Oman Mountains. A sand dune formation covers the aquifer. Within the central model area, the static water level is encountered between 90 m and 107 m amsl. The average thickness of the main aquifer is in the range of 30 m to 50 m. Overlaying dune sands, forming a thick unsaturated zone, cover the main aquifer [GTZ-DCO/ADNOC, 2003]. The lower subunit of the Quaternary sediments represents a fully saturated aquitard, which is situated above the aquiclude, consisting of the Tertiary Lower Fars unit [GTZ-DCO/ADNOC, 2002].

3.3.3 Aquifer system

In the Liwa area the aquifer is unconfined aquifer with sufficient vadose zone thickness around 45 m. Surface materials are highly permeable so as to allow water to percolate easily [Gale et al., 2002]. Sand dunes cover all the Liwa area and water is expected to penetrate easily.

Soil vadose zone is the layer in which desalinated water is injected. The aquifer thickness of the unsaturated zone and the volume of the injected water are related to each other and should be considered carefully. Injection of large amounts of water may exceed the capacity of the unsaturated zone and cause the saturation of the vadose layer. Such case will result in a rapid transmission of water to discharge points that may be formed above the ground surface. On the other hand, considerably thick unsaturated layer may result in water loss through evaporation due to the high temperature in the area.

The investigations explained in this section include the shallow aquifer properties encompassing the resistivity, the electrical conductivity and the total dissolved salts.

1. Resistivity

The resistance [R] per unit length [L] of a surface area [L^2] is in essence the resistance of a cube to the one-way passage of electricity. Apparent resistivity is used in a number of geophysical and hydrogeological applications. Resistivities in the central area of the Western Region are exceptionally good (more 100 ohm.m). The resistivities decrease gradually towards the surrounding areas. In addition, the resistivity decreases continuously and exponentially with the depth. The rate of decrease is constant along several depth intervals.

The vertical resistivity gradient varies, depending on the area. In the Liwa central area, for example, the resistivity gradient is high: 1-3 ohm.m/m. In the surrounding areas it reduces to less than 1 ohm.m/m [GTZ-DCO/ADNOC,2005]. Only the top section of the aquifer formation (whose thickness is constant throughout the area) has a constant resistivity. The gradient in the lateral direction is variable. The resistivity gradient is low in the central Liwa area and becomes high with the distance from the centre [GTZ-DCO/ADNOC, 2005].

2. Electrical Conductivity (EC) of Produced Water

Lower values of electrical conductivities are found in the central Liwa area (between 1,400 μS/cm and 4,100 μS/cm). Intermediate values are found in areas surrounding the central Liwa zone. For example, the electrical conductivity in west Bu Hasa – Baynunah – Dhafra varies between 2,570 μS/cm to 6,175 μS/cm, between 7,600 μS/cm and 21,750 μS/cm in Bu Hamrah – Qusahwira, and between 2,050 μS/cm and 15,270 μS/cm in Madinat Zayed – Asab.

3. Total Dissolved Solids (TDS) contents

Using the conversion factor 0.625, the total dissolved solids (TDS) content can be estimated from the electrical conductivity [Filterswater, 2004]

$$\text{TDS content} = 0.625 * \sigma \text{ (water) in ppm} \qquad (\sigma \text{ in } \mu S/cm) \qquad 3\text{-}1$$

In the Western Region, the TDS content ranges between 800 and 1,300 ppm in the central sub area of Liwa (fresh water). The TDS is higher in the surrounding sub areas, sometimes exceeding 5,000 ppm (medium brackish), like in the eastern sub area of Bu Hurrah [GTZ-DCO/ADNOC, 2005]. Table 3.5 presents the groundwater classification of Abu Dhabi Emirate. Figure 3.4 presents the TDS distribution in the Emirate.

The UAE standards and WHO standards are used to distinguish the water quality, as expressed in part per million [ppm] as Total Dissolved Solid content. Table 3.6 shows the correspondence between the aquifer resistivity, electrical conductivity and TDS. Water is considered fresh when the salt concentration in ppm is less than 1,000 (WHO Standards) or less than 1,500 (UAE Standards). These values correspond to an electrical conductivity of 1600 μS/cm or 2,300 μS/cm and to a resistivity of 40 ohm.m and 28 ohm.m, respectively. For brackish water the upper TDS content is 10,000 ppm and the corresponding EC and resistivity values are 16,000 μS/cm or 11 ohm.m, respectively.

brackish water the upper TDS content is 10,000 ppm and the corresponding EC and resistivity values are 16,000 μS/cm or 11 ohm.m, respectively.

Table 3.5 Summary of groundwater classification schemes used in Abu Dhabi Emirate [EAD, 2005]

Source	TDS range (mg/l)	Classification
Brook et al, 2005a	0-1500	Fresh
	1,500-8,000	Low brackish
	8,000-15,000	High brackish
	15,000-35,000	Saline
	> 35,000	Hyper saline
GTZ/Dornier Consult/ADNOC (2005a)	0-1500	Fresh
	1,500-4,000	Slightly Brackish
	4,000-7,000	Medium Brackish
	7,000-10,000	Strongly Brackish
	10,000-25,000	Slightly Saline
	25,000-50,000	Medium Saline
	50,000-100,000	Strongly Saline
	> 100,000	Brine
USGS/NDC (1996)	0-1500	Fresh
	1,500-15,000	Brackish
	> 15,000	Saline
Forestry Dept Abu Dhabi Municipality & Agriculture	0-1500	Fresh
	1,500-10,000	Brackish
	10,000-20,000	Saline
	> 20,000	Very saline
Abu Dhabi Municipality & Agriculture Agricultural Extension Service	0- 4,000 μS/cm	Class I Fresh
	4,000- 8,000 μS/cm	Class II Low brackish
	8,000 -12,000 μS/cm	Class III high brackish
	> 12,000 μS/cm	Class IV saline
Al Ain Municipality & Agriculture Agricultural Extension Service(2001)	0-1,000	Class I very Fresh
	1,000- 2,000	Class 2 fresh
	2,000-4,000	Class 3 low brackish
	4,000-6,000	Class 4 saline
	6,000- 8,000	Class 5 high brackish
	> 8,000	Class 6 saline

Table 3.6 Correspondence between aquifer resistivity, electrical conductivity and TDS content of produced water in the Liwa area [GTZ-DCO/ADNOC, 2005]

	Slightly Saline	Slightly Brackish		Fresh Water				
				U.A.E Standard		WHO Standard		
Resistivity [ohm.m]	10.6	15.2	20	28.5	30	40	60	100
Conductivity [μS/cm]	16129	6494	4000	2381	2222	1538	952	541
TDS Content [ppm]	10161	4091	2520	1500	1400	969	600	341

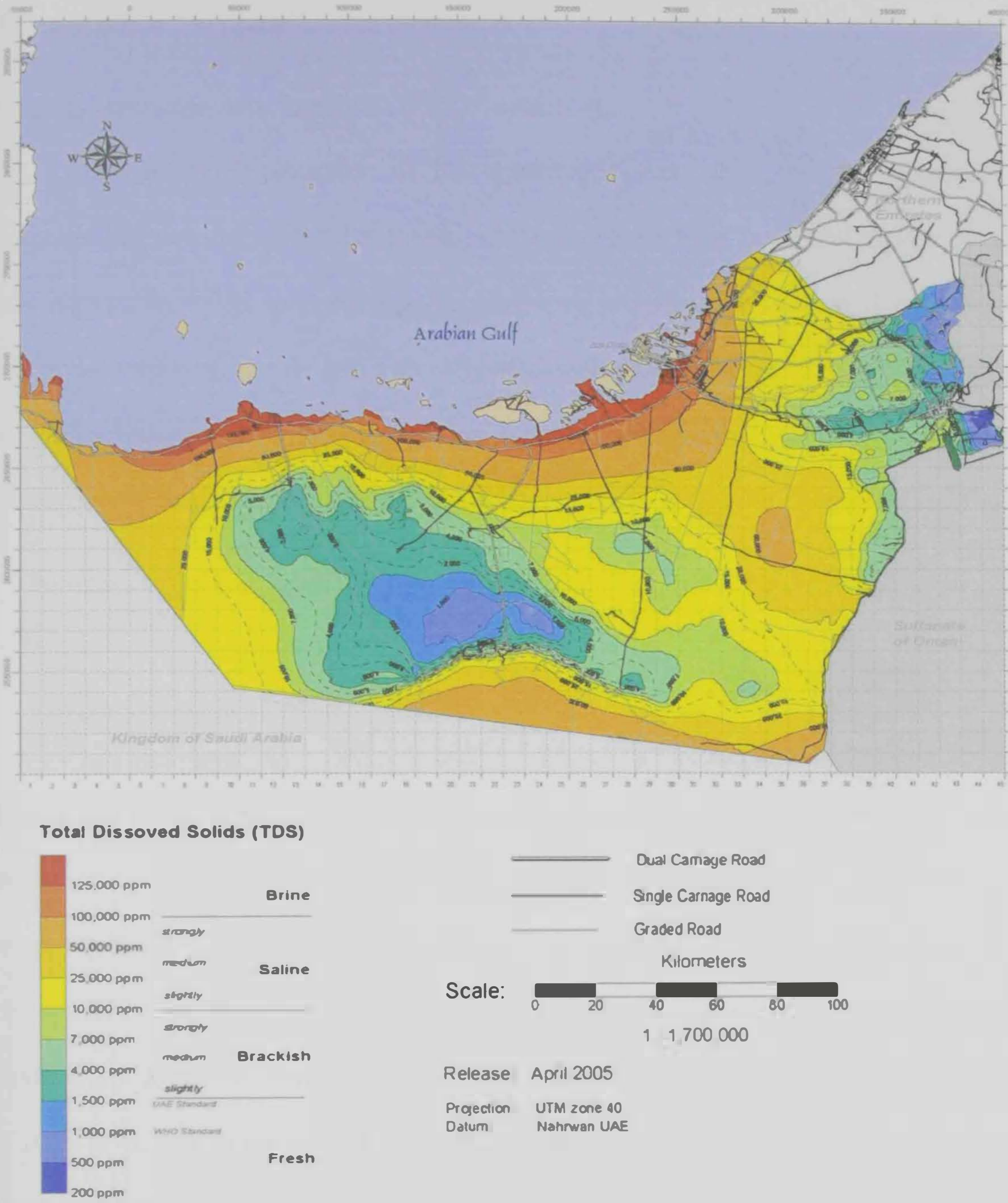

Figure 3.4 Total Dissolved Solid (TDS) of the Emirate [GTZ-DCO/ADNOC, 2005]

3.3.4 Aquifer hydraulic properties

Porosity, transmisivity and hydraulic conductivity

The hydrologeolocial properties of the unsaturated zone determine the suitability of a particular location for artificial recharge. Optimally, areas used for artificial recharge should have high permeability soils, capacity for horizontal movement of water in the saturated zone and in the receiving aquifer and a thick unsaturated zone [GTZ-DCO/ADNOC, 2002]. The volume of water stored in the vadose zone depends on the porosity of the sand. In Liwa area, the porosity is considerably high and ranging between 25- 40% with of an average 35 %.

- Porosity, n (Balke) $= 0.021*(1+\frac{1}{Cu^{2/3}})$ [Hölting,1992] 3-2

- Effective porosity, n_e (Balke) $= n*(1-\frac{Cu^{0.5}}{12})$ [Hölting, 1992] 3-3

Aquifer transmissivity should be high enough to allow for water movement from the mound created around the wells but should not be too high so that water cannot be recovered. Not every aquifer made up of sand dune or lime stone is preferable for recharging. The transmissivity of the aquifer depends on the aquifer thickness and the geohydraulic conductivity. In the study area, it ranges from 100 m²/d to 3,000 m²/d, with an average value of 1065 m²/d. The range of hydraulic conductivity varied between 2 m/day to 106 m/d with an average value of 27 m/d. Geohydraulic conductivity

The following equations are reported in literature and were modified according to the conditions of Abu Dhabi. They were verified with the results of pumping tests. The average value of the hydraulic conductivity was used. The effect of the temperature is added to the equations and is represented by θ in degree Celsius. Table 3.7 shows the average calculated geohydraulic conductivity (k):

- $K_{hazen} = d_{10}{}^2 * 0.0116*(0.7+0.03\theta)$ [Hölting,1992] 3-4 with $Cu \le 5$

Where d_{10} = grain size that is 10% finer by weight (effective grain size) and Cu is the uniformity coefficient = d_{60}/d_{10}

- $K_{ZIESCHANGE} = d_{10}^{2} * 0.0139 * (0.7 + 0.3\theta)$ [Hölting,1992] 3-5

  with $1 \le Cu \le 3$ & $0.1 \le d_{10} \le 0.6$

- $K_{ZIESCHANGE} = d_{10}^{2} * 0.0116 * (0.7 + 0.03\theta)$ [Hölting,1992] 3-6

  with $0.1 \le Cu \le 3$ & $0.1 \le d_{10} \le 0.6$

and

- $K_{BEYER} = d_{10}^{2} * 0.012 * (\frac{d_{60}}{d_{10}})^{-0.2054} * (0.7 + 0.03\theta)$ [Hölting,1992] 3-7

  with $1 \le Cu \le 20$ & $0.06 \le d_{10} \le 0.6$

Storage coefficient and specific yield

Soil samples which were collected from the study area within the activities of the Groundwater Assessment Project-GTZ from different depths. Samples were analyzed for their porosity, according to standard laboratory methods [GTZ-DCO/ADNOC, 2002]. Some formulas were used to calculate the porosity as indicated in equations 3-13 through equation 3-17. An example of the grain-size distribution curve is given in Figure 3.5 and the average estimated value is presented in Table 3.8. The specific yield may also be calculated from the following equations:

- Specific yield (Marotz) $= 46.2 + 4.5 * \ln(k)$ 3-8

- Specific yield (Balke) $= 0.4 + 0.05 * \log(k)$ 3-9

- Specific yield (Bayer) $= (0.0587 * \ln(k) + 1.3463) * n * 100$ 3-10

Table 3.7 Average estimated conductivities according to the formulas (equation 3- 4 through 3- 7)

Well ID	Sample description	Sample Depth m belowground	Hydraulic conductivity [k] m/d
		1.5	27.20
GOW-75		4	26.30
		5	31.26
		7	28.98
		8	17.44
		9	14.01
	Sand dune	10	14.34
		11	17.43
		13.6	20.87
		14.7	19.05
		15.7	20.74
		16.8	20.77
GOW-73	Fine to medium Sand	15.2	33.46
		54.9	10.86
GWA-328		64	32.86
		64	36.96
		50	24.86
	Sand dune	50	26.71
GOW-100		37.5	65.63
GOW-102		48.2	19.92
		69.2	9.68
GOW-103		44.6	22.36
		77.7	26.57
GOW-105		42	32.68
		71.8	16.72
	Core	91.5	14.43
		103.8	93.15
GOW-41		28	15.01
		35	29.61
		45	21.96
		55	11.28
		109	10.94
		140	10.94

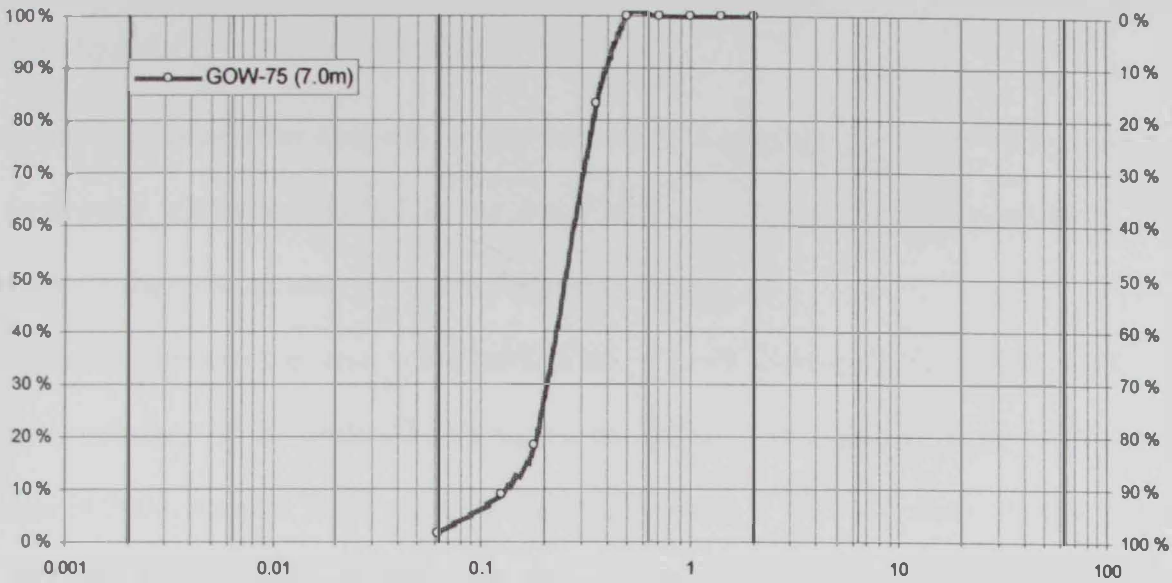

Figure 3.5 Curve for graphical determination of d_n for GOW-75

Table 3.8 Average calculated porosity and specific yield

Well ID	Porosity n_e Balke	Porosity	Estimated Range of Specific Yield (Sy) % Marotz	Balke	Beyer	Average	Sample Depth m-ground
GOW-75	0.34	0.30	9.9	22.5	29.6	20.7	1.5
	0.35	0.31	9.8	22.4	30.1	20.8	4
	0.33	0.29	10.2	22.6	29.2	20.7	7
	0.32	0.27	7.9	21.5	27.0	18.8	8
	0.33	0.29	6.9	21.0	27.8	18.6	9
	0.34	0.30	8.3	21.7	28.8	19.6	14.7
	0.34	0.30	8.7	21.9	29.3	20.0	15.7
	0.34	0.30	8.7	21.9	29.3	20.0	16.8
GOW-73	0.34	0.30	10.8	22.9	30.0	21.3	15.2
	0.30	0.26	5.8	20.5	24.9	17.1	54.9
GWA-328	0.32	0.28	10.8	22.9	28.6	20.8	64
	0.33	0.28	11.3	23.2	29.1	21.2	64
	0.34	0.30	9.8	22.5	29.8	20.7	50
GOW-100	0.34	0.30	13.9	24.4	31.8	23.3	37.5
GOW-102	0.30	0.25	8.5	21.8	25.7	18.7	48.2
	0.27	0.21	5.3	20.2	22.0	15.8	69.2
GOW-103	0.31	0.26	9.0	22.1	26.6	19.2	44.6
	0.35	0.31	9.8	22.4	30.2	20.8	77.7
GOW-105	0.35	0.31	10.7	22.9	30.6	21.4	42
	0.28	0.22	7.1	21.1	23.0	17.1	91.5
	0.25	0.18	15.5	25.2	23.9	21.5	103.8
GOW-41	0.32	0.28	7.2	21.2	27.0	18.5	28
	0.35	0.32	10.3	22.7	31.1	21.4	35
	0.35	0.31	6.0	20.6	28.5	18.4	55
	0.32	0.27	5.8	20.5	26.0	17.5	109
	0.32	0.27	5.8	20.5	26.0	17.5	140

3.3.5 Aquifer native water quality

Native and recharge water qualities have to be addressed carefully. Existence of large volume of freshwater with good quality in the study area was one of the main reasons for the selection of the recharge area [Rizk and Alsharhan, 2003].

Liwa aquifer contains the fresh water basin north of Liwa Crescent [USGS/NDS, 1993] and is the beneficiary of the artificially recharged desalinated water introduced as a pilot ASR scheme in 2004. Another fresh water mound is also found in the dune sands of the Bu Hasa oil field [Rizk and Alsharhan, 2003]. In the Liwa area, where 2,400 km^2 are underlain by fresh groundwater [Moreland, 1998]. High quality water should not be injected into poor groundwater system. Where desalinated water is to be stored, it is usually sufficient to show that it meets the international standards [David and Pyne, 1995].

Table 3.2 and Table 3.3 show the quality of desalinated water that was used.

Furthermore, due to high quality of the desalinated water, no major geochemical compatibility problems are expected [Mukhopadhyay et al., 1998]. The desalinated water can be treated to minimize any potential reactions with the aquifer materials; for example the pH can be adjusted to be non- aggressive [Bouwer, 2002].

3.4 Hydraulic Gradient

The lateral movement of water should be as low as possible to store water for future use. This criterion is very important for recharge projects. Spreading of the stored water to other locations where water can not be recovered should be avoided. In the study area, the lateral movement is in range of 10 m/y, which is acceptable for ASR. The hydraulic gradient is relatively small.

3.5 Groundwater Flow

The Western Region of Abu Dhabi Emirate is separated by a major divide, running approximately from east to west. It passes the greater Liwa area 20 km north of the Liwa Crescent. All shallow to medium deep – seated groundwater north of it flows to the north, towards the Arabian Gulf. The groundwater south of the divide flows to the south toward Saudi Arabia. The huge Sabkha areas which are much lower in elevation than the central part of the Western region function as a discharge area, where tremendous groundwater volumes constantly evaporate. The study area is located exactly along this major groundwater divide. It encloses a pronounced local groundwater dome with highest groundwater table elevation within the Western region. The measured groundwater level is between 103.6 m and 107.2 m amsl, with an average of 106.2 m amsl. Figure 3.6 shows the contour map of groundwater level in Liwa. The groundwater flow takes place from the highest head in the central part of the eastern half of the study area. The groundwater flows radialy to the adjacent areas under a maximum hydraulic gradient of about 0.5%.

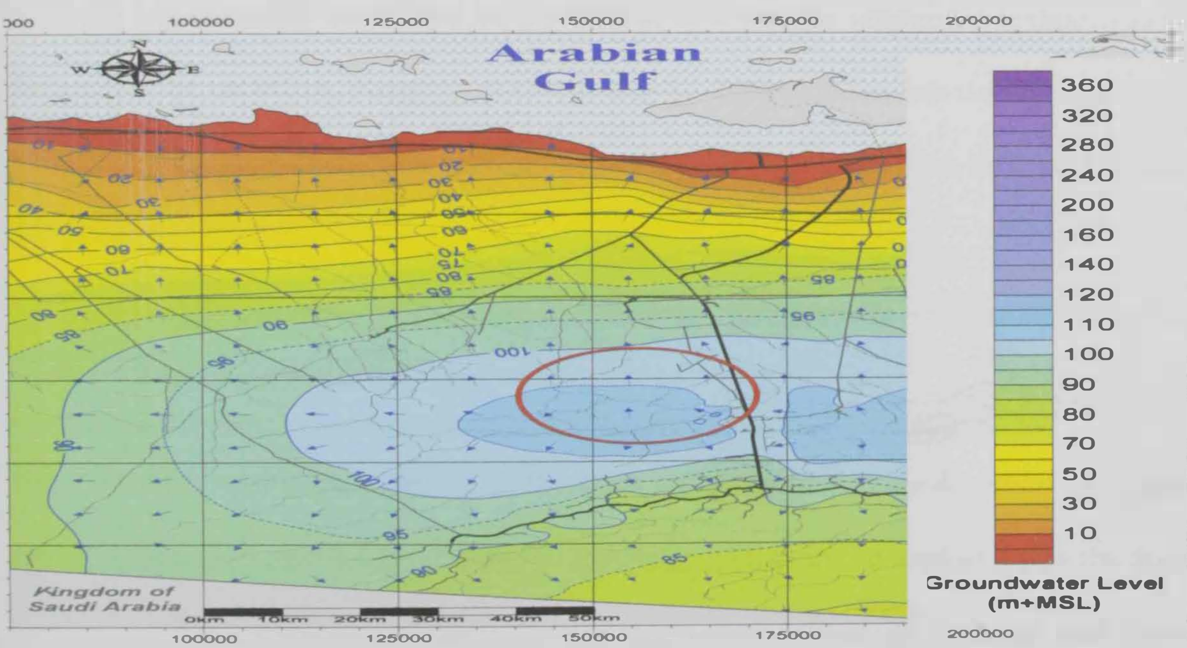

Figure 3.6 Contour map of groundwater levels and the velocity vectors

3.6 Overview of Conducted Field Tests and Laboratory Analysis

The following are some of the field tests and laboratory analyses that have been carried within the activities of the Groundwater Assessment Project- GTZ. The results of these tests are used in the modelling of the hydrogeological system using a groundwater numerical model (FEFLOW).

- **Pumping tests**: four pumping tests have been carried out in order to collect well and aquifer-related data. These data include, among others, geohydraulic conductivity, transmissivity, storage coefficient and well performance characteristics.
- **Field infiltration tests**: Infiltration tests were conducted to obtain information related to infiltration rates and vertical geohydraulic conductivity.
- **Grain size analyses**: Soil samples were collected and analyzed and grain size distribution curves have been determined from the samples that were taken from different depths. These analyses were also used in the calculations of geohydraulic conductivity, porosity and specific yield using empirical formulas.

3.6.1 Pumping tests

Pumping tests are preformed to determine the characteristics of aquifers and wells. The pumping test is usually conducted for a period of time and the changes in hydraulic heads in the aquifer are observed. Pumping rates can also be used to determine the capacity of wells. Many pumping tests have been conducted within the study area and detailed information regarding the followings parameters was obtained:

- Geohydraulic parameters (including transmisivity, geohydraulic conductivity and storage coefficient of the aquifer).
- Well performance characteristics (well capacity and well efficiency).

The evaluated parameters established the basis for all necessary geohydraulic calculations, especially for the numerical groundwater modelling, which can be used to assess the possible impacts on the groundwater regime in the study area caused by recharge and recovery activities. The estimated values of the transmissivity and geohydraulic conductivity that were derived from the long-term pumping tests are presented in Table 3.9. The pumping tests revealed a transmisivity range between 100 m²/d and 3,000 m²/d, with an average value of 1,067 m²/d and a median of 950 m²/d. The estimated geohydraulic conductivity varies between 2 and 106 m/d with an average value of 27 m/d and a median of 28 m/d, (Table 3.10).

Table 3.9 Estimated values of transmissivity and geohydraulic conductivity for the study area [GTZ-DCO/ADNOC, 2002]

Well Identification	Transmissivity (T)		Geohydraulic conductivity (k)		Infiltration
	[m²/d]	[m²/s]	[m/d]	[m/s]	m/d
GWA-141	1050	1.2E-02	19	2.2E-04	
GWA-144	250	2.9E-03	6	6.9E-05	
GWA-148B	500	5.8E-03	13	1.5E-04	
GWA-151	900	1.0E-02	22	2.5E-04	
GWA-153	830	9.6E-03	16	1.9E-04	
GWA-156	800	9.3E-03	28	3.2E-04	
GWA-164	245	2.8E-03	6	6.9E-05	
GWA-172	140	1.6E-03	5	5.8E-05	
GWA-177	1200	1.4E-02	45	5.2E-04	
GWA-178	1500	1.7E-02	38	4.4E-04	
GWA-215B	2500	2.9E-02	51	5.9E-04	
GWA-240	950	1.1E-02	28	3.2E-04	
GOW-65	1200	1.4E-02	36	4.2E-04	
GOW-68	2000	2.3E-02	62	7.2E-04	
GOW-70	1600	1.9E-02	48	5.6E-04	
GOW-71	3000	3.5E-02	106	1.2E-03	
GOW-72	250	2.9E-03	67	7.8E-04	16.32
GOW-73	1500	1.7E-02	36	4.2E-04	18.5
GOW-74	700	8.1E-03	14	1.6E-04	
GWA-311	400	4.6E-03	15	1.7E-04	
GWA-313	950	7.5E-03	28	3.2E-04	
GWA-317	100	1.2E-03	2	2.3E-05	
GWA-320	750	8.7E-03	9	1.0E-04	
GWA-321	2000	2.3E-02	28	3.2E-04	
GWA-322	1850	2.1E-02	28	3.2E-04	
GWA-327	500	5.8E-03	8	9.3E-05	12.5
GWA-328	1100	1.3E-02	31	3.6E-04	11.8
GWA-330	1100	1.3E-02	30	3.5E-04	

Table 3.10 Summary of geohydraulic conductivity from pumping test

	Pumping Test	
	Transmissivity (T) [m^2/d]	Geohydraulic conductivity (k) [m/d]
Average	1067	29.5
Max	3000	106
Min	100	2
Median	950	28

Examples of pumping test calculations

The step-drawdown test was run using four to five steps of increased discharge rates with a duration of three hours each. After termination of pumping, the recovery of the water level was recorded until the initial static groundwater level was reached. In each step, measurements and calculations were carried out as follows:

- Maximum Discharge, Q_n [m^3/h]: The maximum discharge was measured in each time step.
- Drawdown, Sob [m]: The drawdown was measured in each time step and the calculations included the following components, S_A, S_i and S_w. The draw down is calculated as $S_{ob} = S_A + S_i + S_w$ where the definition for each component is described below:

Drawdown related to recharge distance [m] , S_i

$$S_i = S_{ob} - S_{corr} \quad \text{3-11}$$

Where S_{corr} is the corrected drawdown

Drawdown related to aquifer properties [m] , S_A

$$S_A = \frac{Q_n \;-\; Hydraulic\ Rsistivity}{3600} \quad \text{3-12}$$

Draw down related to well losses [m], S_w

$$S_w = \begin{cases} = 0 & if \quad S_{ob} - S_i - S_A < 0 \\ = S_{ob} - S_A & if \quad S_{ob} - S_i - S_A \geq 0 \end{cases} \quad \text{3-13}$$

- Specific capacity [m^2/h]: The specific capacity of a production well is calculated as

$$Specific\ Capacity = \frac{Q_n}{S_{ob}} \qquad 3\text{-}14$$

- Specific drawdown [m] = $\frac{S_{ob}}{Q_n}$ 3-15

Electrical Conductivity, EC [µs/cm]: The electrical conductivity, EC was measured in each time step. Figure 3.7 shows the discharge versus the electrical conductivity. The EC increases as the discharge is increased.

- Temperature [C]: The water temperature was measured in each time step. Figure 3.8 shows the discharge versus the water temperature. The temperature of the pumped water decreases as the discharge increase.

- Transmisivity, T [m^2/d]: An empirical relation between specific capacity and transmisivity was used as follow [Freeze and Cherry,1979]

$$Transmisivity(T) = 15.3\left[\frac{(Qn * 24)}{S_{ob}}\right]^{0.67} \qquad 3\text{-}16$$

- Aquifer Thickness, H [m]: The aquifer thickness was estimated from available boreholes.
- Hydraulic conductivity, k [m/d]: The hydraulic conductivity is calculated as

$$Hydraulic\ Conductivity(k_f) = \frac{T}{H} \qquad 3\text{-}17$$

Where T is the transmisivity (m^2/d) and H is the aquifer thickness in m.

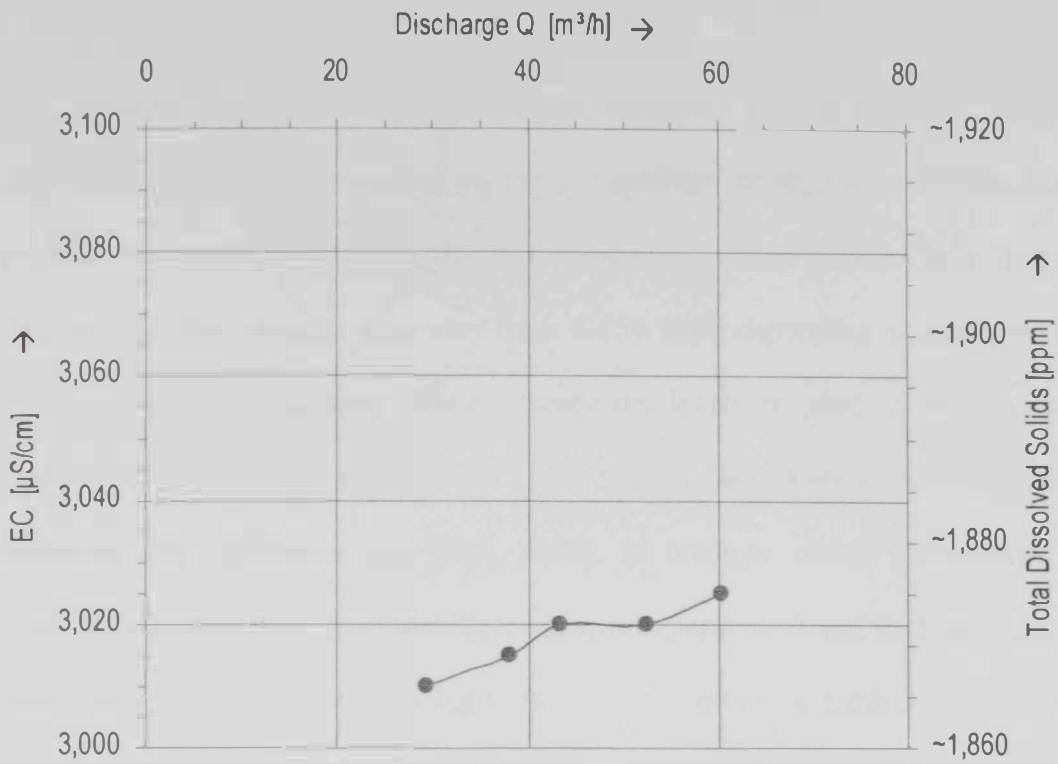

Figure 3.7 Discharge [m^3/h] versus electrical conductivity [μS/m] and TDS [ppm]

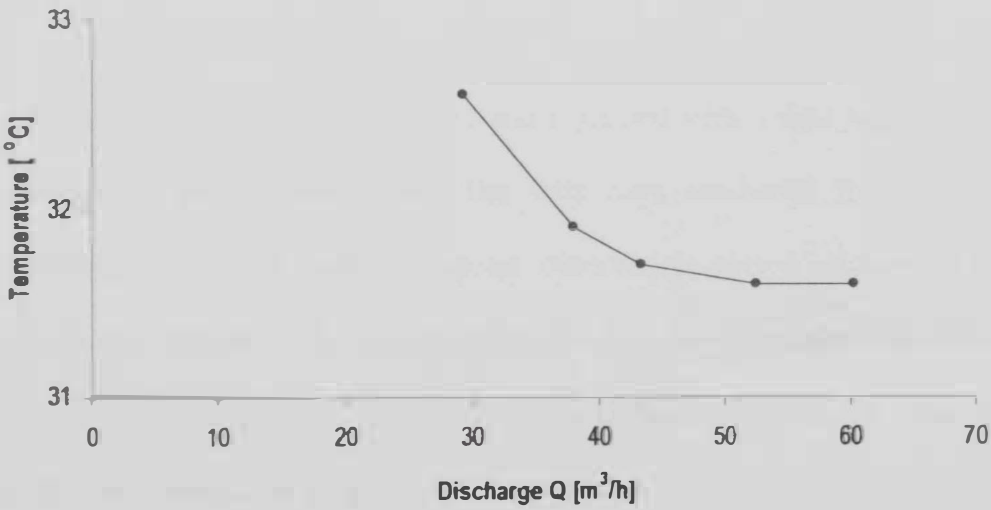

Figure 3.8 Discharge (m^3/h) versus temperature (C)

3.6.2 Field infiltration tests

Infiltration rates are function of many parameters including the soil hydraulic conductivity and the development of the mounding on the groundwater table. Average infiltration rates must consider the cyclic operation of the recharge basins, which includes both wetting and drying periods. Infiltration rates may vary from 8-150 cm/d depending on soil type and the development of the clogging layer. When groundwater levels are shallow, mounding below the recharge basin can increase groundwater levels to near the bottom of the infiltration basin thus decreasing the infiltration rate [Fox, 1999]. In order to obtain information on the infiltration rates and vertical geohydraulic conductivity, one well and four tank infiltration tests were conducted during the pumping tests of some wells (see Table 3.9). A bottomless, approximately $12m^2$ steel tank (4.89 m [L] x 2.44 m [W] x1.24 m [H]) has been constructed with a pipe, sealed on one side and connected via a flexible hose and steel pipes to a pumping well. Inside the tank, a pressure sensor and a backup system for groundwater level measurements and a Multi-Parameter Sensor (MPS) for electrical conductivity and temperature measurements were installed. Continuous records were obtained in order to monitor any changes in groundwater levels and quality. Figure 3.9 shows two piezometers near the infiltration tank that were installed and equipped with a data logger to record any possible changes of groundwater levels. The tests were conducted for two days with a constant discharge from the wells. After an observation period of several hours, the infiltration rate was adjusted. The dynamic water levels in the infiltration tank increased to a certain level and then started to decline, indicating a higher infiltration rate. This is probably caused by the final extrusion of air from the porous medium in the vadose zone by the infiltration water, leading to a water-saturated zone with an increased hydraulic conductivity. The vertical hydraulic conductivity in the vadose zone ranged between 1.6×10^{-4} m/s and

2.5×10^{-4} m/s, indicating an approximately 49 % lower value than that of the horizontal hydraulic conductivity. The results of the infiltration tests are given in Table 3.11.

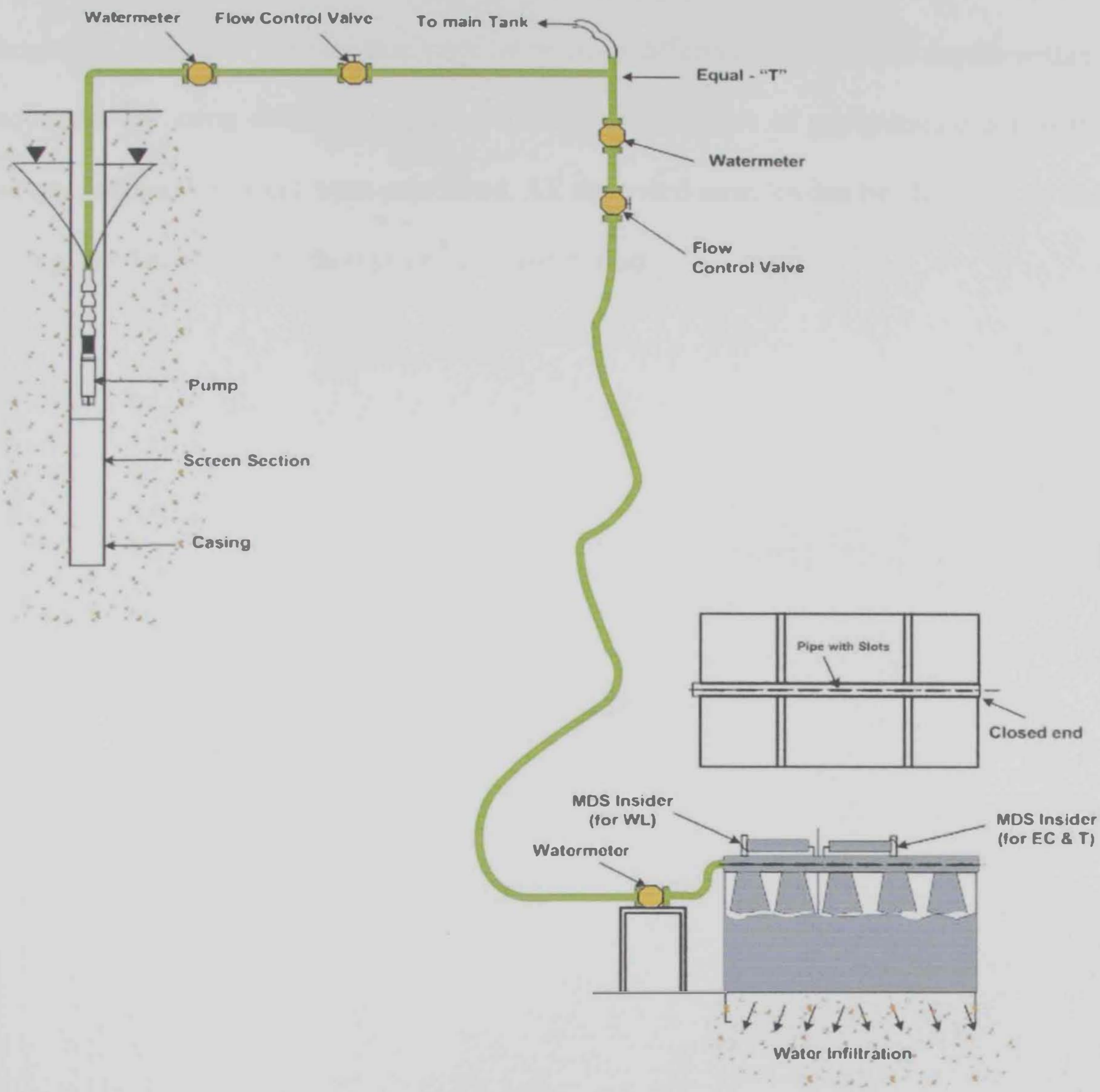

Figure 3.9 A general layout of Infiltration test

Table 3.11 Results of infiltration tests [GTZ-DCO/ADNOC, 2002]

Well	GOW-72	GOW-73	GWA-327	GWA-328
Average Infiltration rate (m/h)	0.68	0.77	0.52	0.49
Average water infiltration volume through tank (m^3/h)	8.1	9.2	6.2	5.2

3.6.3 Grain-size analysis

Grain size analysis by sieving provides a useful and a relatively simple method for estimating some geohydraulic properties of aquifer. Therefore, grain size distribution curves have been determined from sand samples that were taken from different locations and depths within the study area. By using different empirical formulas, the values of geohydraulic conductivity, porosity and specific yield were calculated. All the tested samples can be classified as fine to medium sand according to their grain size distribution.

Chapter IIV

Groundwater Modelling

4 Groundwater Modelling

An accurate groundwater model requires a huge amount of information about the study area. The general steps in developing a groundwater model include: (1) developing the conceptual model, (2) defining the model domain and the software to be used, (3) calibrating and verifying the model and (4) making predictions [Richard, 2002]. The conceptual model represents how the aquifer responds to different excitation. Developing a good conceptual model requires compiling detailed information on the geology, water quality, recharge, water levels, hydraulic parameters and pumping. The selection of a model domain depends on which computer program is used and the dimensions of the layers and cells that make up a model. Calibration and verification demonstrate the model ability to reproduce water levels measured in the past.

A good calibration and verification gives high confidence that the developed model produces reasonable predictions of future water levels. After completing the calibration and verification of a numerical model, it can be used to make good predictions.

In the following sections the process of groundwater modelling is presented. Generally, the objective of designing the correct groundwater flow model is to create a tool for testing methods of different groundwater injection applications and estimating the impact on the hydraulic regime in the vicinity of the study area. This step is, among others, essential prior to the study of any contamination problem. The methodology and main steps of groundwater modelling, description of the model area and boundary conditions are presented in this chapter.

4.1 Methodology

Groundwater models are developed in several stages as summarized hereafter.

4.1.1 Preparing conceptual model

A study domain with a total area of 50,000 km^2 was considered. The domain is bounded by the Arabian Gulf confining the model area from the northern border. The Sabkha Matti, where groundwater evaporates, defines the western border of the model. The eastern border is defined from the Qusahwira area to the coast. The southern border lies south of the Liwa Crescent and extends in the east-west direction. Based on the hydrogeologic setting and boundary conditions of the area under consideration a conceptual model has been created as given in Figure 4.1. The model consists of 3 layers; unsaturated zone, saturated aquifer, and an aquitard. The upper unsaturated layer allows for aquifer recharge through rainfall and excess of irrigation water. The bottom layer (aquitard) does not allow for any gain or loss through it.

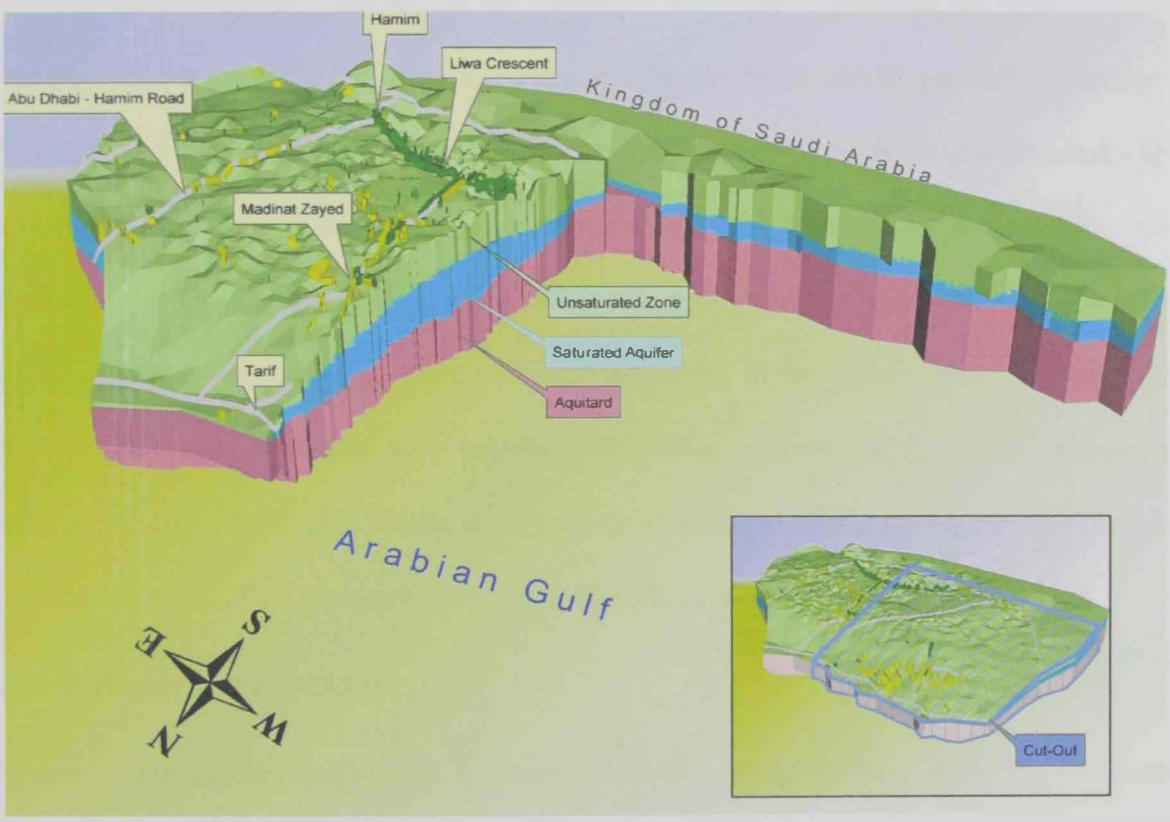

Figure 4.1 Conceptual model of the study area

4.1.2 Model extent and geometry

A 3D steady state flow model has been considered to describe the mainland in the Western Region. It covers an area of about 50,000 km², being roughly described by a rectangle of the following coordinates (UTM Zone -40):

Upper left = x: -3,000 y: 2,683,000

Upper right = x: 297,000 y: 2,683,000

Lower left = x: -3,000 y: 2,471,000

Lower right = x: 297,000 y: 2,471,000

The geometry of the three-dimensional groundwater flow model describes the Quaternary unit of the local stratigraphy. The Quaternary unit is divided into two subunits. The upper subunit forms the partially saturated aquifer while the lower subunit can be characterized as a fully saturated aquitard. The top of the Tertiary Lower Fars formation, an aquiclude, forms the base of the groundwater model. Consequently, the basic model geometry consists of 3 layers, where the upper two layers divide the aquifer to an upper homogeneous and a lower, layered part. On the third layer, the characteristics of the aquitard are assigned. In order to create 3 layers, 4 so-called slices are needed, which define the borders between the adjacent layers. The top slice represents the surface topography of the model area. The second slice defines the border between two aquifer sections in accordance to the interpretation of lithologic logs of boreholes inside the model area. The top of the Lower Fars represents the base of the model. The system in Western Region is shown in Figure 4.2 .

4.1.3 Boundary conditions

Boundary conditions define the interaction of the study domain with the surrounding environment. Correct selection of boundary conditions is a critical step in a model design. Boundary conditions are mathematical statements specifying the dependent variable (head) or the derivative of the dependent variable (flux) at the boundaries of the study domain

[Anderson and Woessner, 1992]. Setting boundary conditions is the model design step that is most subject to serious error.

The northern border of the study domain is assigned as first type boundary condition, prescribing the fixed hydraulic head of the Arabian Gulf (head of zero value is assigned along the border). Figure 4.3. The western and southern borders are regarded as no-flux boundaries. Only along the eastern border, specified flux boundaries (NEUMANN) have been set. They define a small influx of water from the Eastern Region into the model domain.

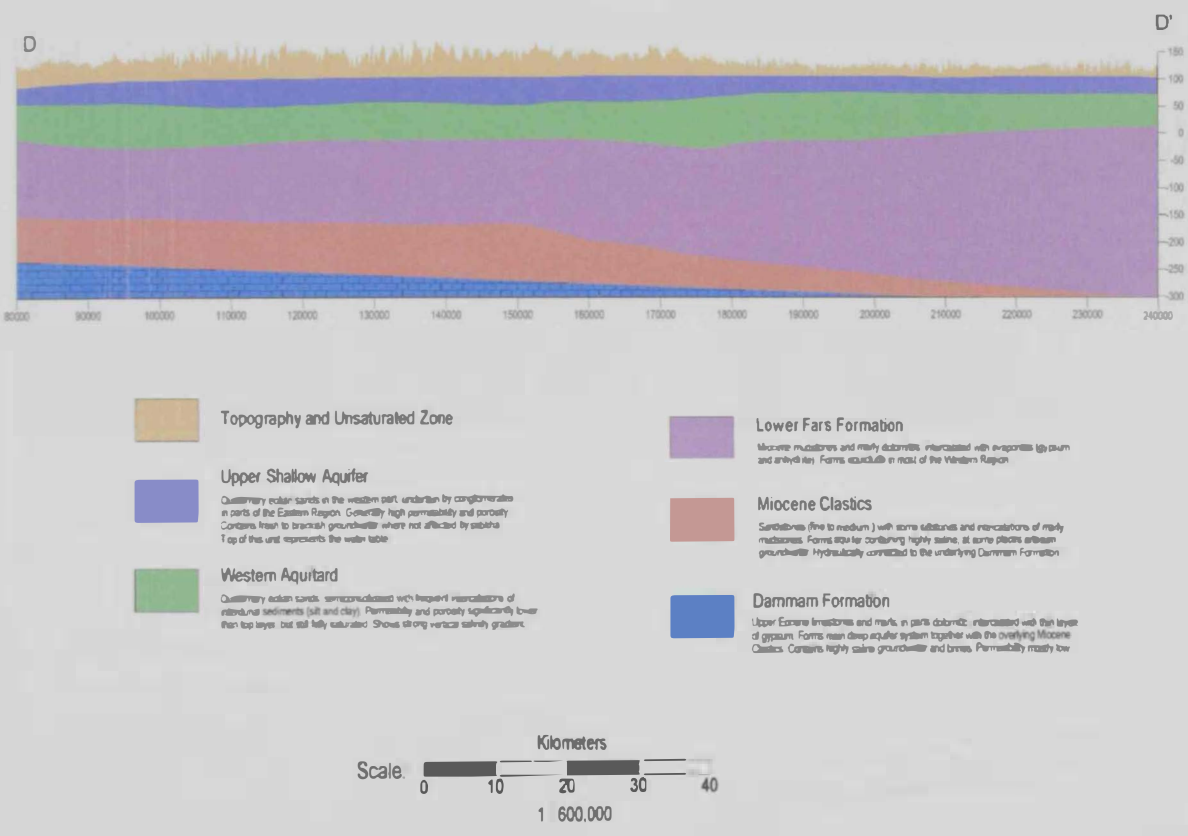

Figure 4.2 Layered system in Western Region

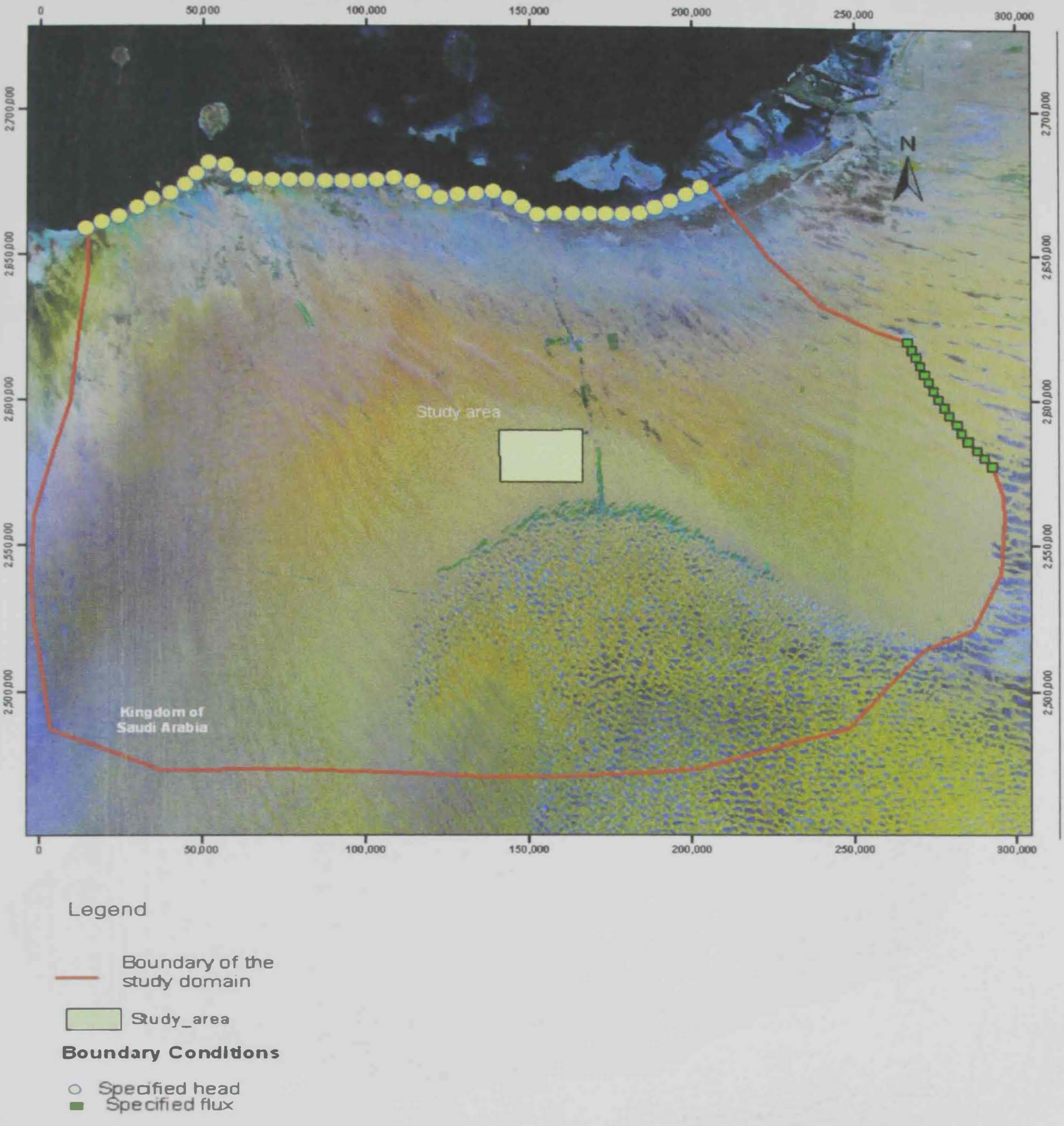

Figure 4.3 Study domain and boundary conditions

4.1.4 Mesh design

Triangular elements were selected for the discretization of the study domain. An automatic mesh generation was chosen with 1000 elements at the initial stage. Refinement of mesh was achieved with the help of GIS where the selected location to be refine was inserted into FEFLOW as a background map and then the refinement process was applied. The total number of elements was 277,008 with 173,340 nodes as shown in Figure 4.4 .In the study area the element size was less than 10 m.

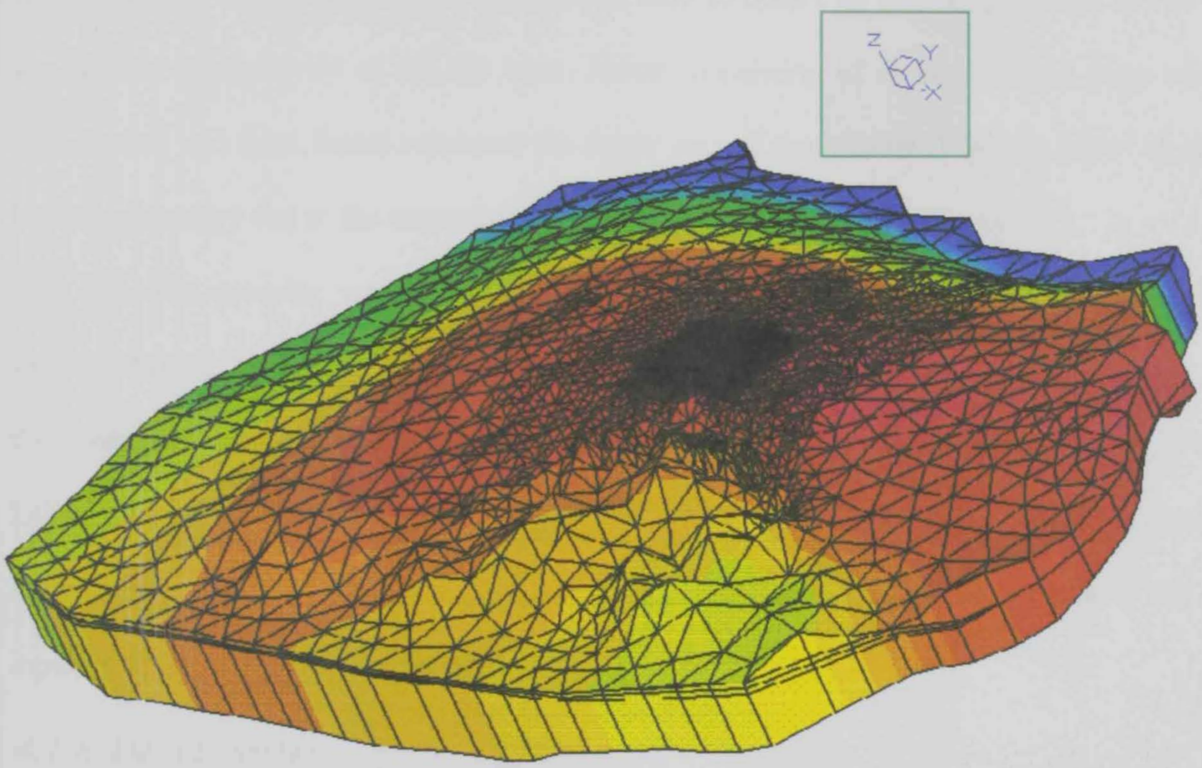

Figure 4.4 3D Model geometry

4.1.5 Data preparation

The data and other relevant information were collected, analyzed and prepared to be used in the simulation package (FEFLOW) and Geographic Information Systems (GIS). This was a time consuming task. Data of Satellite Imagery, land use data, groundwater levels, properties of different layers were collected and reviewed for its consistency. Data were mostly obtained from the reports of Groundwater Assessment Project- GTZ and related database. Kriging method, a geostatistical analysis technique, was used to interpolate data and obtain some missing information. Examples of the data prepared for the model is given in Appendix – I.

4.1.5.1 Hydraulic parameters

Information related to the central model area (artificial recharge area) and the hydrogeologic settings of the aquifer, which are represented by the three uppermost layers, were available. The horizontal hydraulic conductivity is quite high. Values from 5×10^{-5} m/s to 8×10^{-4} m/s were reported. The analysis of the infiltration tests indicates an anisotropy factor of 0.4 for the vertical conductivity of the top layer, mostly consisting of unconsolidated dune sands. The second and third layers represent the lower part of the aquifer. The two layers show a higher anisotropy due to the more extensive layering. Consequently, a factor of 0.1 is applied. The effective porosity was set to 20 % as attained by geohydraulic field tests [GTZ-DCO/ADNOC, 2002]. A horizontal conductivity of 5×10^{-6} m/s to 1×10^{-5} m/s is assigned for the bottom layer (aquitard). A vertical anisotropy factor of 0.05 is taken [GTZ-DCO/ADNOC, 2002]. An effective porosity of 5 % seems suitable for the aquitard. For the surrounding areas less information is available. Reference is made to Appendix II for the input of data of the hydraulic parameters.

4.1.6 Data transfer

The interface between the FEFLOW and GIS was very helpful in preparing, converting and transferring data from GIS to FEFLOW and vice versa. GIS is a computer-based tool for mapping and analyzing data and events. It integrates common database operations such as query and statistical analysis with the unique visualization and geographic analysis benefits offered by map [ESRI, 2006]. ArcMap version 9.1, which is the premier application for desktop GIS and mapping, was used. ArcMap helps to visualize the data geographically and provides all the tools needed to present the data on a map and display it in a clear manner.

4.1.7 Extending the 2D model to a 3D model

Starting the simulation with 2D was an essential step to test the functionality of the numerical model. Figure 4.4 shows the model geometry. The model was then extended to 3D and the mesh geometry was also developed in a 3D domain. Therefore, all datasets for initial head distribution, boundary conditions and hydrogeological parameters were re-imported to FEFLOW using the implemented regionalization technique of Kriging.

4.1.8 Steady state modelling

The focus of the groundwater flow model is set on the Northern Liwa area with its known relatively huge groundwater resources and its fresh water lens, where the artificial recharge takes place as shown in Figure 4.5. A steady-state model for groundwater flow was developed and calibrated on the natural flow regime. The model was also validated thereafter. The model behaviour with respect to a different data set has been tested and analysed. Finally, the model can be used for the study of contamination problems. It should be noted, however, that the groundwater flow model does not focus on the depletion of the aquifer system by farming activities.

4.1.9 Calibration of the model

The model calibration is the process in which one or more model parameters (for example, the transmisivity) is adjusted until the model simulations match the independent observations. The process consists of changing values of input parameters in an attempt to match field conditions within some acceptable criteria. The calibration process typically involves calibration with the steady-state conditions where there are no observed changes in hydraulic heads with time for the conditions being modelled. A model calibration should include comparisons between model results and field observations for hydraulic head data [Anderson and Woessner, 1992].

During the calibration under the steady-state conditions, the parameters of uncertain accuracy were adjusted within meaningful margins, until a reasonable match with groundwater levels and flows provided by Groundwater Assessment Project-GTZ was achieved. The aim of the calibration is to reflect the dynamics of the natural groundwater hydraulic regime and to create a reliable tool for testing methods of artificial recharge in Liwa area. Although efforts have been devoted to accurately calibrate the model, some variation occurred specially in the area where groundwater abstraction is possible. However, the groundwater measurements in large parts of the model domain show stable conditions, due to moderate agricultural development and partially the high salinity of the groundwater, which reduced the groundwater consumption. Therefore, a steady-state approach is considered first.

In order to calibrate the model a set of 162 groundwater level measurements for the year 2003 has been considered. Figure 4.6 shows the location of the selected wells. During the calibration period, small changes in the recharge distribution especially in Liwa area and limited variations in the hydraulic conductivities were necessary to achieve satisfactory results as shown in Table 66.1 (appendix –III). Figure 4.7 presents the measured groundwater levels for the year 2003, while Figure 4.8 shows the difference between the measured and the calculated levels. The resulting hydraulic heads of the calibrated model fit well with the measured values at the observation wells. The frequency plot in Figure 4.9 visualises the differences between simulated and measured groundwater levels. About 62% of the data fall in the range of less than 2-3 m difference and about 7% of the data has a difference of more than 5 m. The maximum difference between measured and simulated groundwater levels is about 6.3 m, with a mean of about 2.10 m.

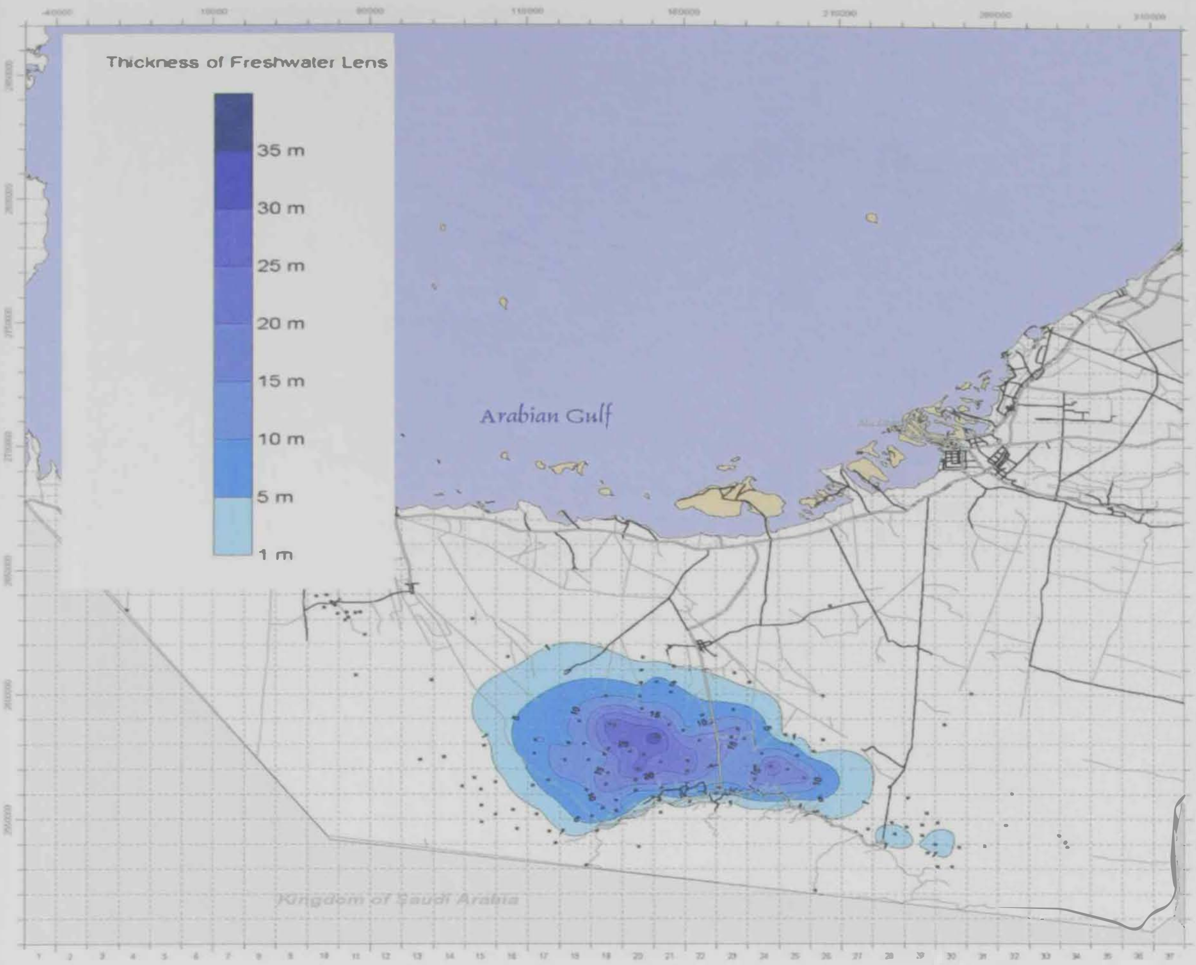

Figure 4.5 Thickness of fresh water lens in the Western Region [GTZ-DCO/ADNOC, 2005]

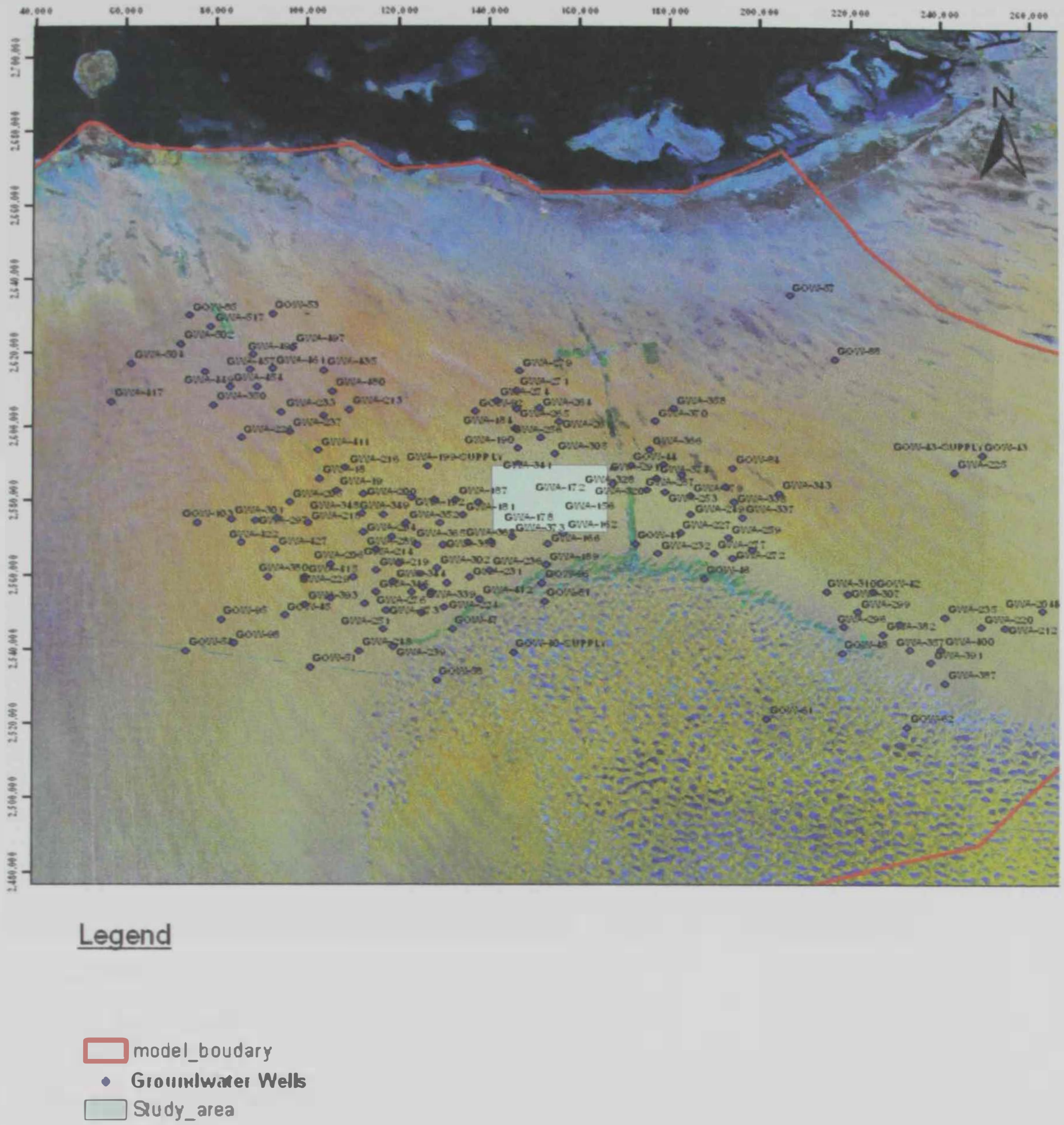

Figure 4.6 Wells used for the calibration of the model

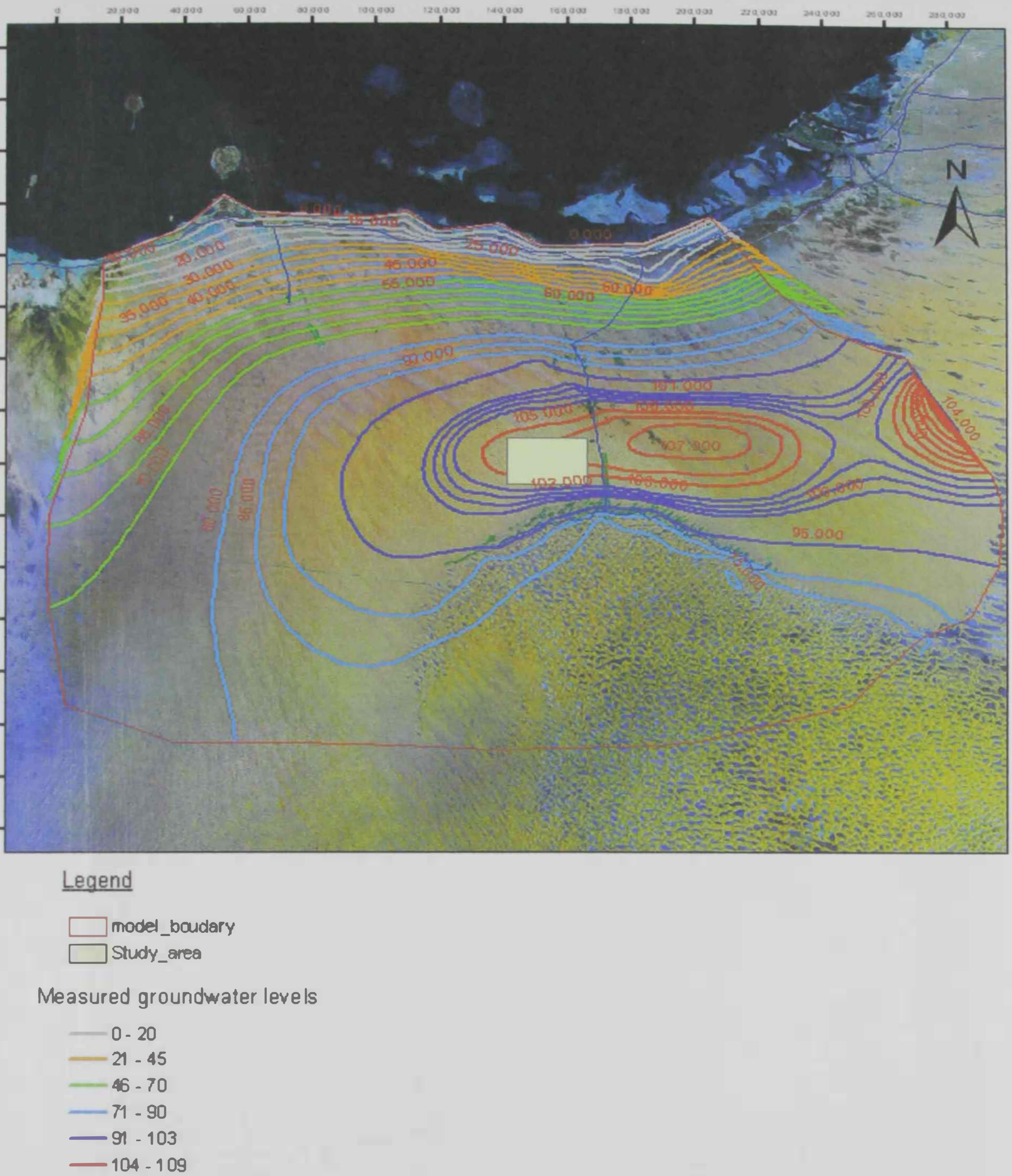

Figure 4.7 Measured groundwater levels (m), 2003

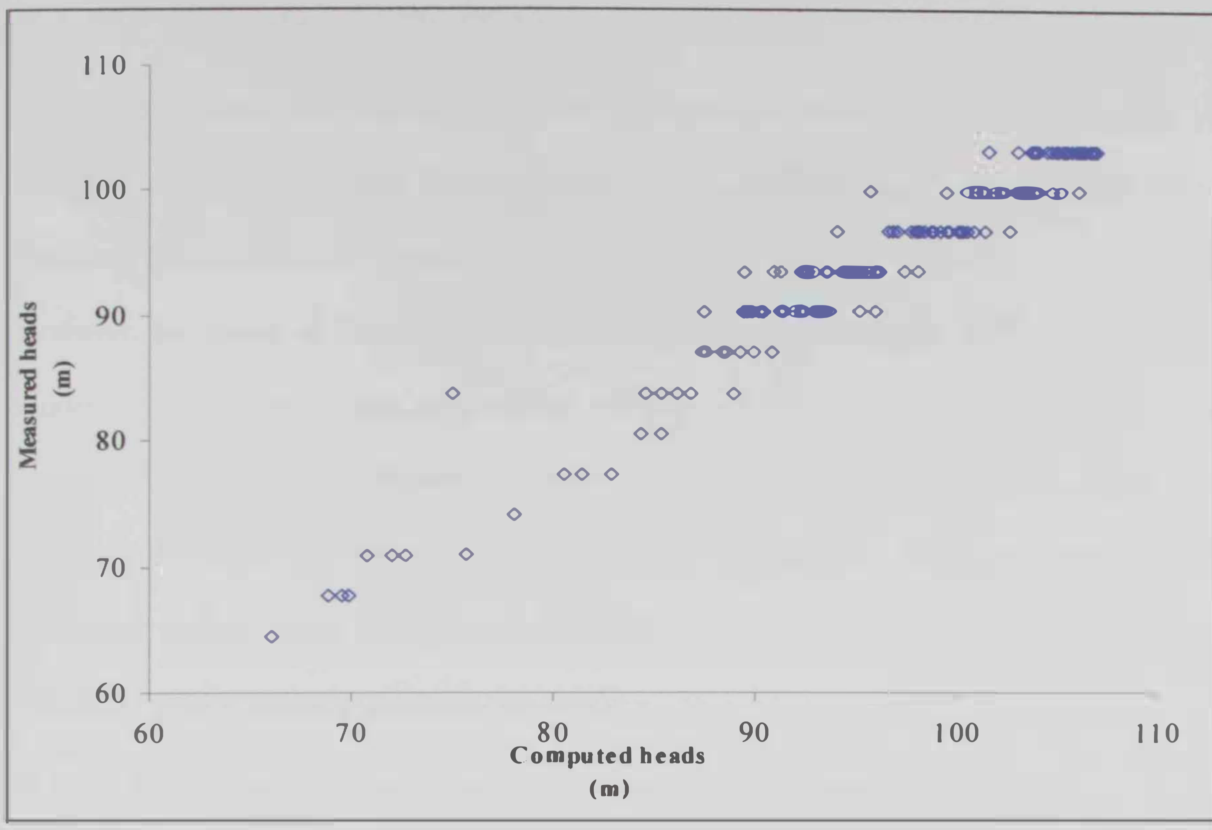

Figure 4.8 Simulated versus measured heads (m) used in the calibration process

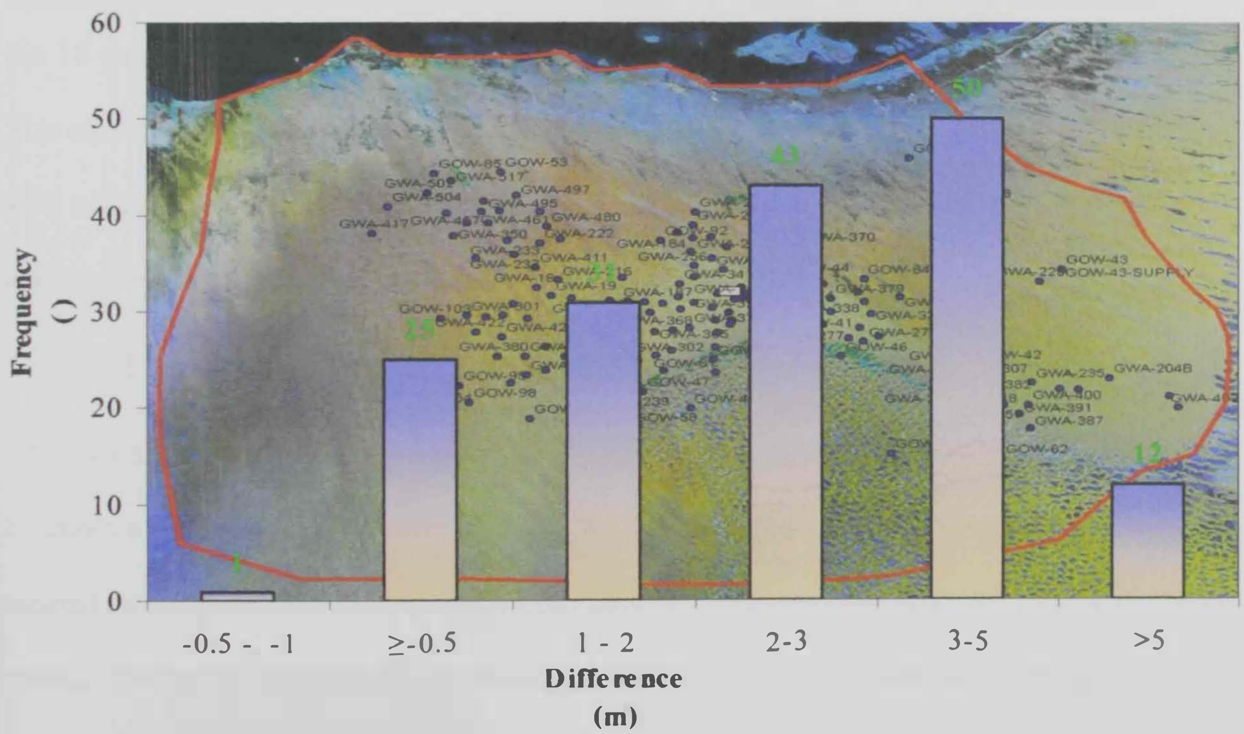

Figure 4.9 Differences between simulated and measured head (m), model calibration

4.1.10 Validation of the model

A calibrated model uses selected values of hydrogeologic parameters, sources and sinks and boundary conditions to match field conditions for selected time periods (either steady-state or transient) [Anderson and Woessner, 1992].

However, the choice of the values parameters and boundary conditions used in the calibrated model is not unique. In this project, two different sets of data were used to examine the behaviour of the model. Validation of the model is an essential step to ensure its accuracy. In this study the validation is done using two different sets of data to confirm the reability of the developed model. In the first scenario the data of the hydraulic head distribution was used and in the second scenario pumping test data was employed.

4.1.10.1 First scenario: hydraulic head distribution

This step was done using 18 groundwater wells as given in Table 4.1. The location of the selected wells is shown in Figure 4.10. Testing the match between simulated water levels and the 18 points of measured water levels was considered to assess the accuracy of the model. The mean of the difference between the measured and simulated levels was about 0.76 m with a maximum difference of 2 m. Figure 4.11 shows the differences between the measured and the simulated groundwater levels.

4.1.10.2 Second scenario: pumping test

The validation process involves using the model to simulate a groundwater configuration, for example a pumping test, for which proper field data exists. In this step the model is verified against an independent observations (in this case, drawdown in the main well and observation wells). The agreement between the model prediction and the observed data is checked.

Introducing stresses on the groundwater system was conducted through two pumping tests for wells GWA-526 and GWA-487. The tests were carried out within the activities of

Groundwater Assessment Project-GTZ. These wells were assigned separately with the pumping rates that have been used in the experiments. The location of the two wells is given in Figure 4.12. Refinement of the mesh was necessary at this step. Therefore, 180,500 nodes and 310,101 elements for the case of well (GWA-526) and 185,900 nodes and 320,150 elements for the case of well (GWA-487) were used to minimize the difference and match the real data and location of the well. The initial and final groundwater levels are represented in Figures 4.14 and 4.15, respectively.

Table 4.2 summarizes the results of the validation of the model. Abstraction rates of 2,688 m^3/d and 2,064 m^3/d were assigned in FEFLOW to reach a drawdown of 14.09 m in well GWA-526 with a difference of 2.91 m than the measured level. For the well GWA-487 a difference of 1.1 m between simulated and observed heads was observed. Three observation wells for each main well were installed at different distances as in the field to have more accurate results. For the well GWA-526 the distance of the observation wells varies from 15 to 60 m away from the main well. As the distance increases the difference between the measured and calculated head decreases reaching a 0.2m in some cases. The same conclusion was observed for the well GWA-487 with distances of observation wells varying between 25 and 67 m.

The results of the simulation of well GWA-526 show a drawdown difference between the measured and the simulated levels ranging between 0.2 to 0.44 m while for the well GWA-487, the difference ranges from 0.02 to 0.1 m.

Table 4.1 Wells used in the validation of the model

Well ID	X40	Y40	Measured Head [m]	Computed Head [m]	Difference [m]
GOW-63	147,603	2,584,525	106	105.5	0.5
GOW-64	150,423	2,587,950	105	105.5	-0.5
GOW-65	156,702	2,586,672	106	105	1
GOW-66	155,594	2,583,213	107	105.2	1.8
GOW-67	162,281	2,586,101	106	105.5	0.5
GOW-68	150,915	2,585,584	102	105.5	-3.5
GOW-69	154,019	2,580,581	106	105.1	0.9
GOW-70	163,570	2,578,101	107	105.5	1.5
GOW-71	154,019	2,580,581	106	105.5	0.5
GOW-72	163,570	2,578,101	107	105.2	1.8
GOW-73	159,402	2,578,129	107	105	2
GOW-74	164,508	2,581,581	108	106.5	1.5
GOW-75	164,559	2,585,207	108	106	2
GOW-76	164,036	2,575,929	108	107	1
GOW-77	135,504	2,578,406	105	105.5	-0.5
GOW-78	135,927	2,588,198	106	104.5	1.5
GOW-79	146,491	2,576,064	107	106.1	0.9
GOW-80	152,057	2,552,865	93	92.3	0.7

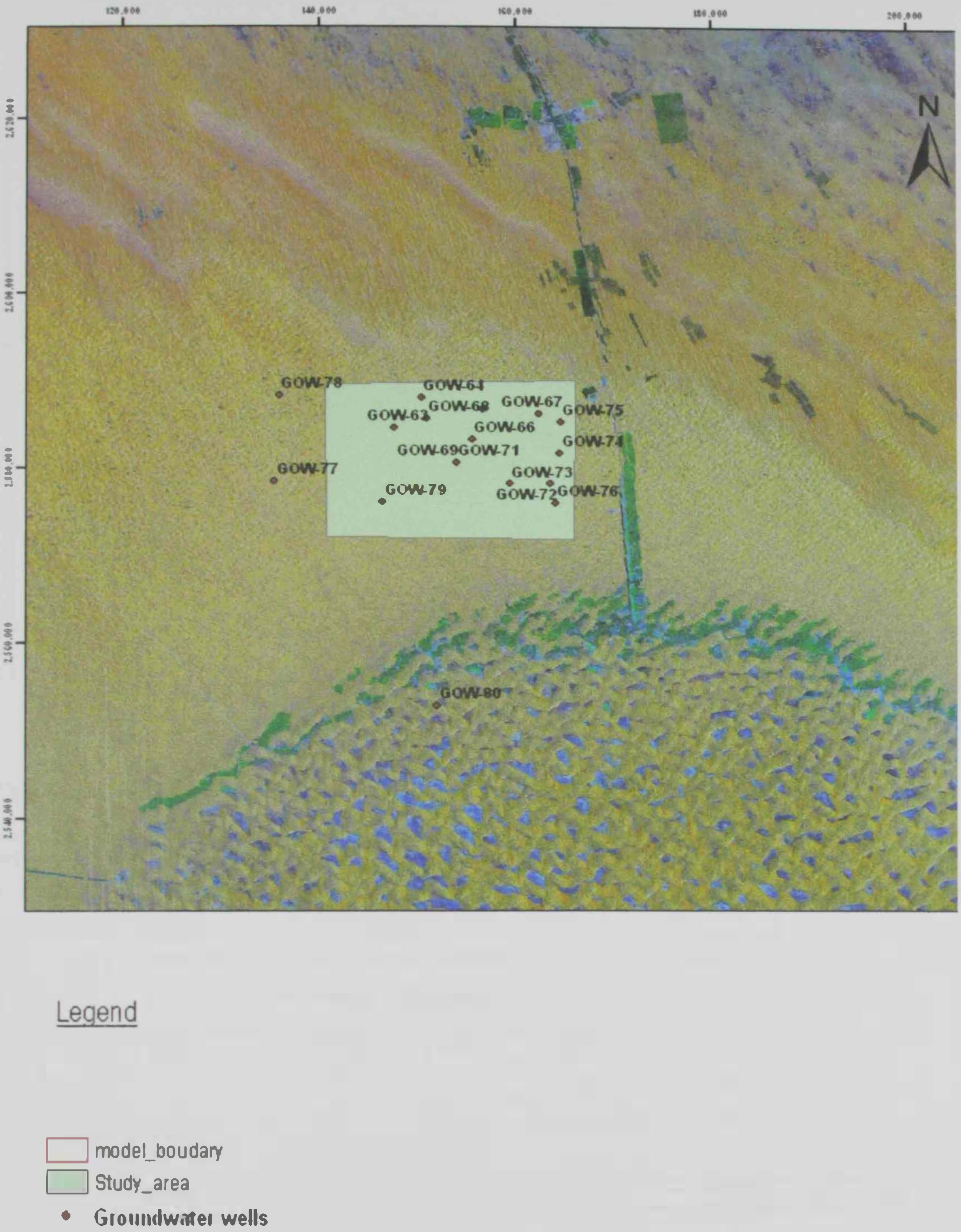

Figure 4.10 Wells used in the validation of the model, first scenario

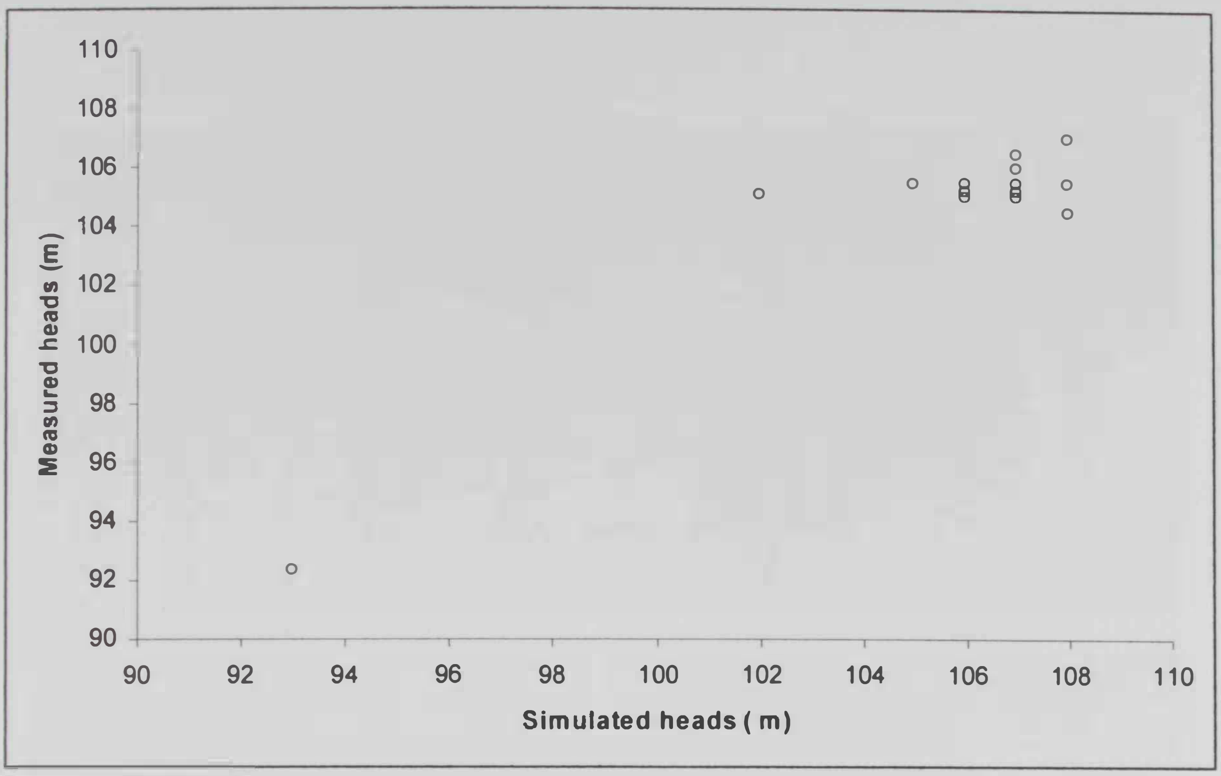

Figure 4.11 Simulated versus measured heads (m) used in the validation process

Table 4.2 Validation results of the model using pumping tests, second scenario

Pumping well	Main well			Observation well			
	Abstraction rate $[m^3/d]$	Drawdown [m]		Well ID	Distance from main well [m]	Draw down [m]	
		Measured	Simulated			Measured	Simulated
GWA 526	2688	17	14.09	P-1	15	1.5	1.72
				P-2	30.4	0.95	1.19
				Supply well	59.5	0.54	0.74
GWA 487	2064	14.5	13.4	P-1	25	1.28	1.11
				P-2	45	0.68	0.66
				Supply well	67	0.41	0.31

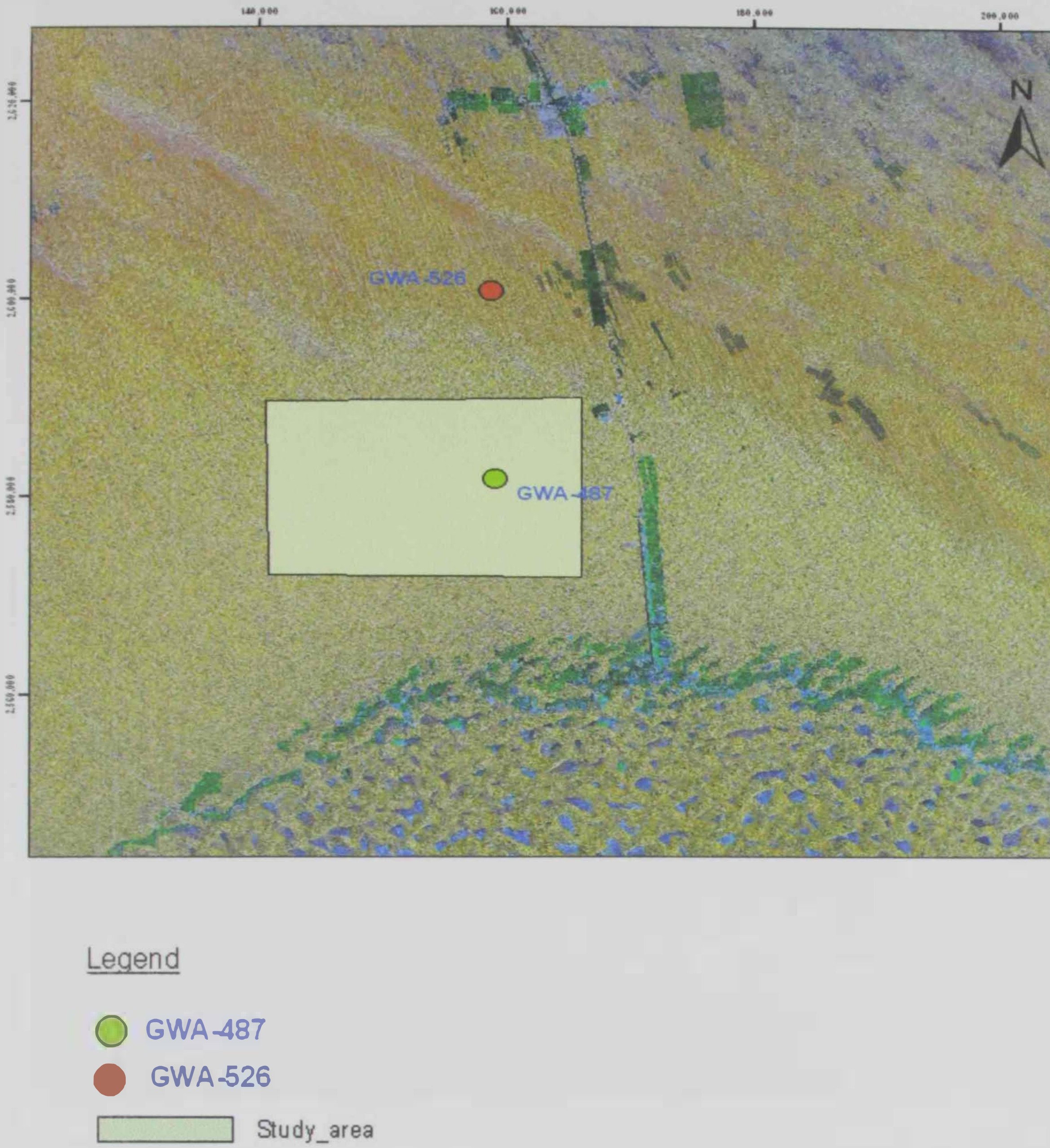

Figure 4.12 Wells used for the validation of the model, second scenario

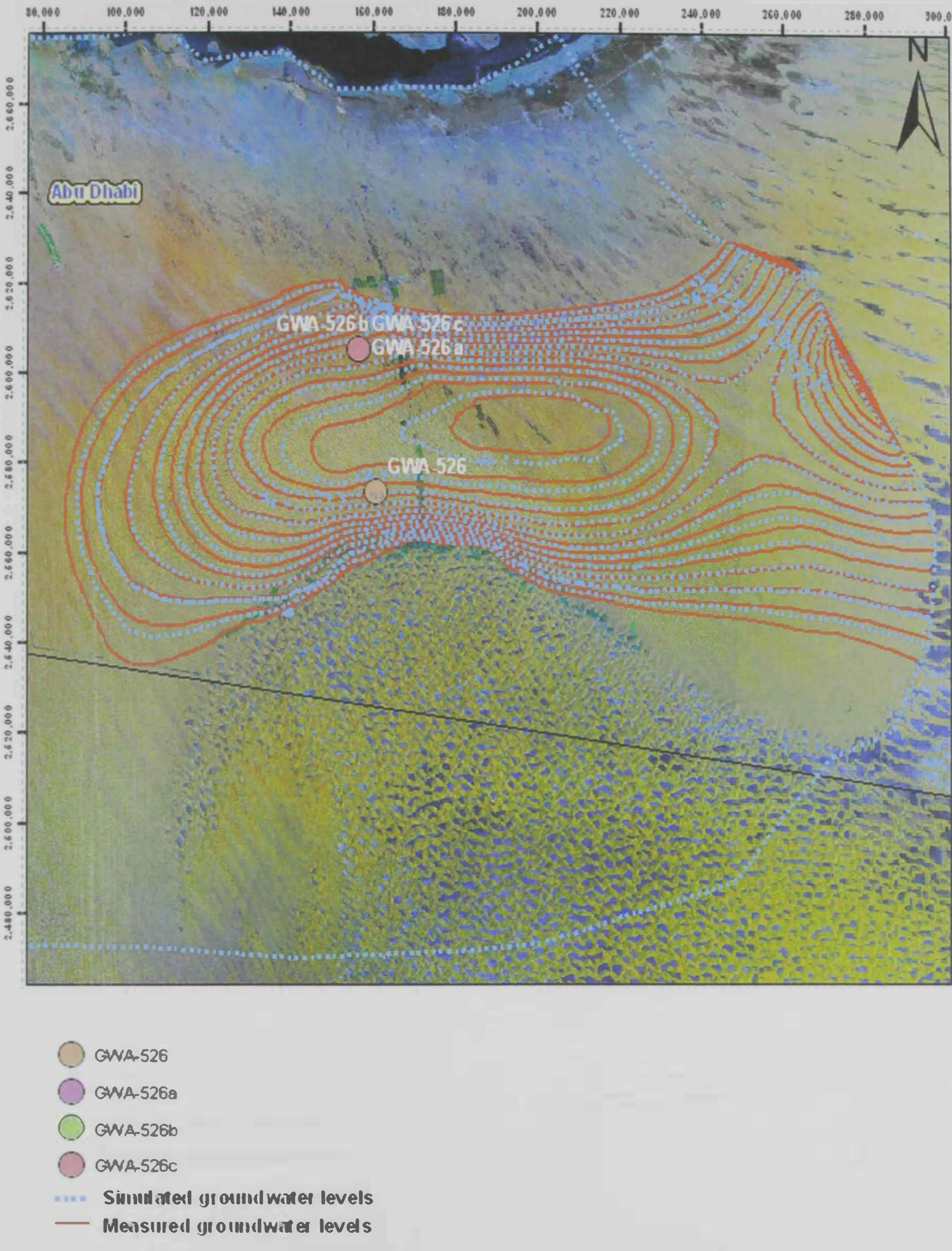

Figure 4.13 Comparison between measured and simulated groundwater levels, GWA-526

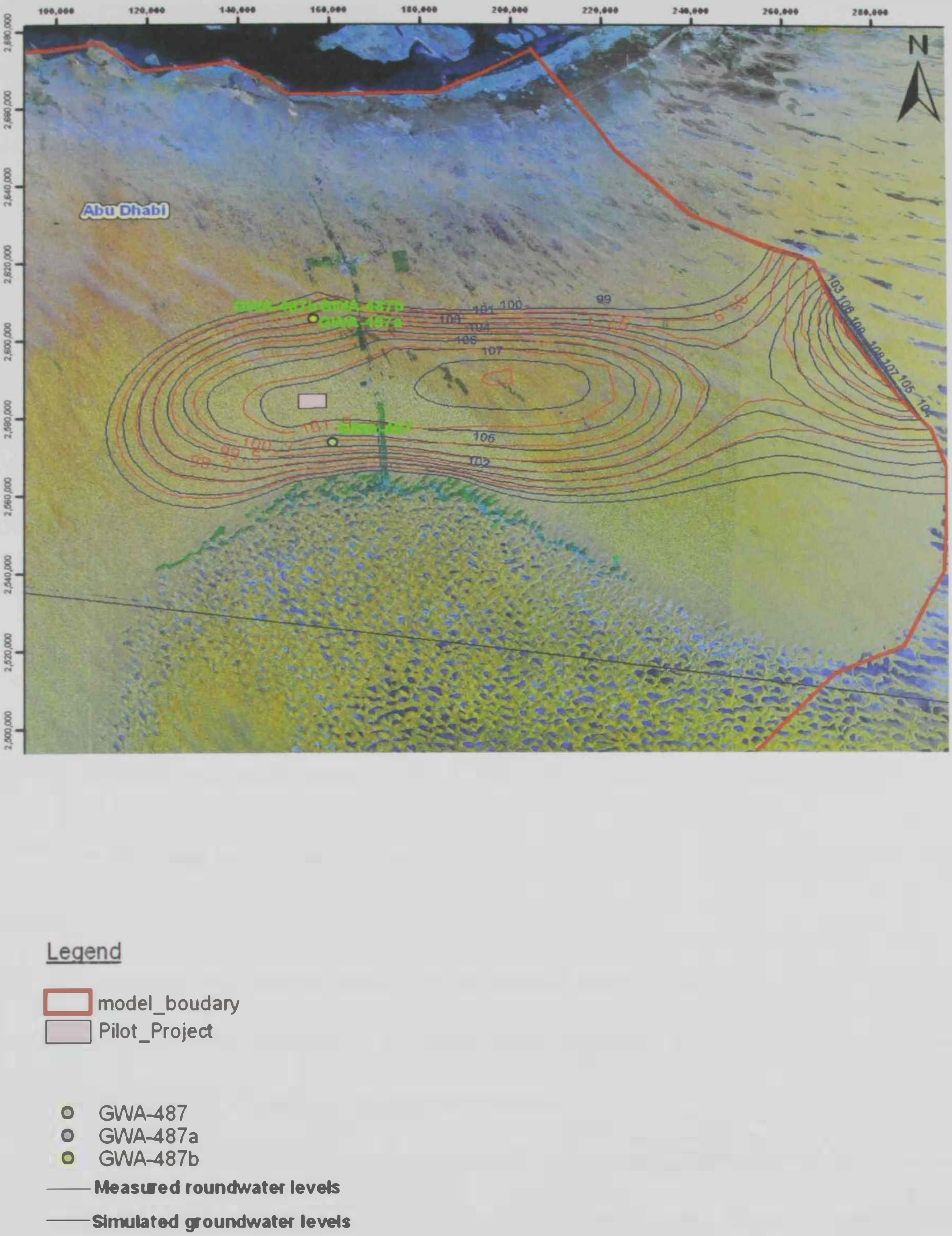

Figure 4.14 Comparison between measured and simulated groundwater levels, GWA-487

4.2 The Groundwater Balance

The natural hydraulic regime in the study area is mainly controlled by three factors:

1. Existing groundwater mound from the Asab region to the Northern Liwa area, creating a groundwater divide in east-west direction.
2. Discharge to the north into the coastal sabkhas (evaporation) and the Arabian Gulf.
3. Discharge to the south into the sabkha flats of the Southern Liwa area, where evaporation takes place, too.

In addition to the above, a discharge to the west into the Sabkha Matti occurs. The natural groundwater balance could be described by the following equations:

$$Q = F_{in\ Oman} - F_{out\ KSA} - F_{out\ ArabianGulf} + R_{preci} - Evap_{Sabkahs} = 0$$

Where, Q = Groundwater balance, $F_{in\ Oman}$ = inflow from Oman boundary, $F_{out\ KSA}$ = outflow to Saudi Arabia, $F_{out\ ArabianGulf}$ = outflow to Arabian Gulf, R_{preci} = Recharge by precipitation and irrigation and $Evap_{Sabkahs}$ = Evaporation from Sabkahs.

Table 4.4 quantifies the balance terms for the natural steady-state groundwater regime as resulted from Fluid Flux analyzer in FEFLOW which estimates the cross sectional flow by the projection of Darcy velocity on the selected sections.

In order to get a steady-state balance a relative high amount of groundwater recharge has to be assumed. Table 4.3 and Figure 4.15 are exported from FEFLOW. Figure 4.15 shows the fluxes into and out of the model domain where the red circles in indicate the in-flow and the blue ones indicate the out-flow. Zero flux is shown as black dot. Different sizes of circles indicate high or low values of fluxes. For example, the eastern border represents the in-flux from Oman and is of small magnitude as compared to the blue circles of the out-flux to the

Arabian Gulf. Some out-flux is shown in the area of Sabkha Matti which is defined as a discharge area. Since the in-flow from eastern border is very small, thus all the natural groundwater recharge in Liwa area comes from precipitation with the red circles. The main area for groundwater recharge is the area of Liwa Aquifer where the groundwater level is about 110 m amsl.

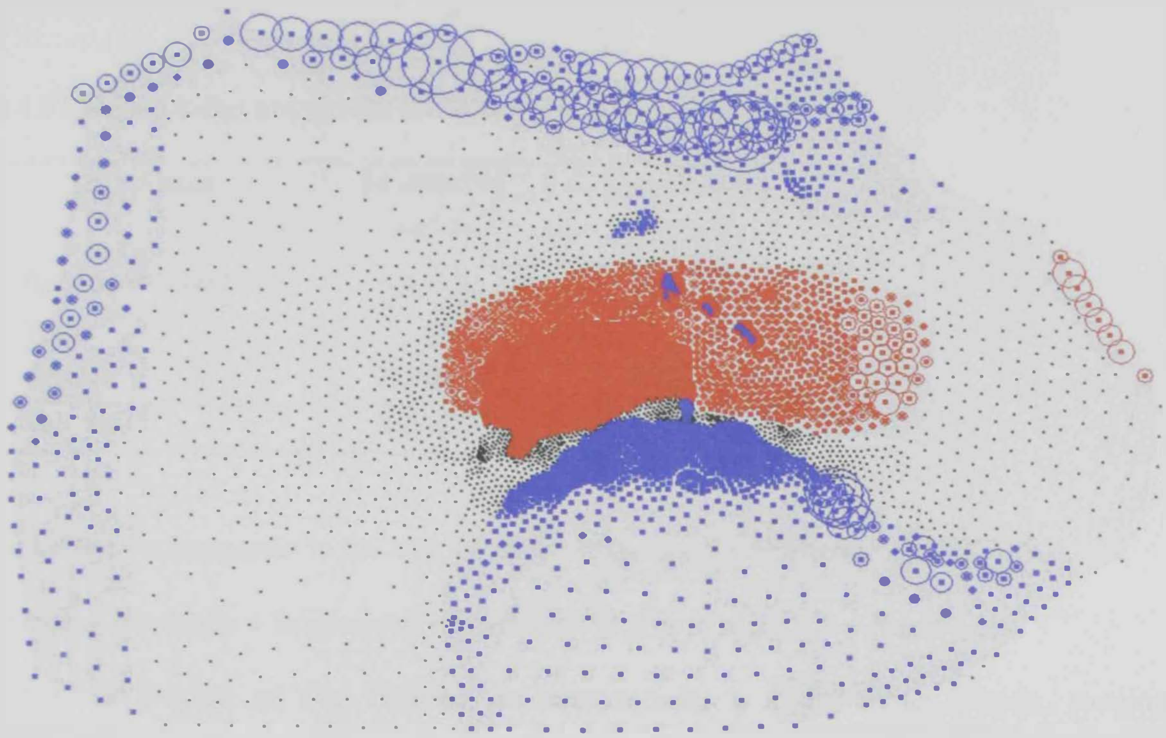

Figure 4.15 In-flux and out-flux in the model domain (steady state)

The total flux into the study domain at steady state is given in Table 4.3. The budget analyzer tools compute the fluid and contaminate mass quantities due to the boundary conditions and the areal fluxes (recharge) assigned. It is an accurate tool that could be used to determine the fluxes due to boundary conditions or due to areal excitations.

The computed quantities for the fluid and contaminate mass are:

- ☐ Fluxes along the outer or inner boundaries, .i.e, border occupied with Dirchlet, Neumann or Cauchy boundary conditions (1^{st} , 2^{nd} and 3^{rd} kind boundary conditions)
- ☐ Injection and withdrawals through single wells (4^{th} kind boundary condition).

- ☐ Areal fluxes due to infiltration/recharge
- ☐ Imbalance: Gain (+) / Loss (-).

The in-flux is estimated as 99,944 m^3/d, of which about 4,436 m^3/d flows through the boundaries and 97,903 m^3/d are received from rainfall and irrigation. The total out-flux is estimated as 99,940 m^3/d, of which about -1,101 m^3/d are lost through the boundaries and 91,635 m^3/d are pumped.

Table 4.3 Resulted in-flux and out-flux of the model [m^3/d]

Fluxes	In -flux (+) [m^3/d]	Out -flux (-) [m^3/d]
Boundary Conditions	4,436.81	-1,101.2
Well	0	0
Areal	97,903.66	-91,635.51
Imbalance	4.299707	0

The imbalance corresponds to the sum of all the influxes and outfluxes. That is

Imbalance = Border-in + Border-out + Well-in + Well-out +Areal-in + Areal-out.

The Western Region of the Emirate is separated by a major water divide, running approximately from east to west. It passes the greater Liwa area, 20 km north of the Liwa Crescent. All shallow to medium deep – seated groundwater north of the divide flows to the north, towards the Arabian Gulf as the receiving body, while all the groundwater south of it flows to the south to Saudi Arabia. The huge Sabkha areas in the south which are much lower in elevation than the central part of the Western Region function as a discharge area, where tremendous groundwater volumes constantly evaporate.

The study area is located exactly along this major groundwater divide. Moreover, it encloses a pronounced local groundwater dome with highest groundwater table elevation within the Western Region. Figure 4.16 shows the groundwater dived and the natural flow, while Figure

4.17 shows the natural direction of the groundwater flow as resulted from the model under the steady state conditions.

Table 4.4 Calculated groundwater flow balance

Balance Term	Quantity [10^3 m^3/d]	Area [km^2]	Mean areal flow [mm/y]
Natural Groundwater Recharge	97.9	4850	7.37
Evaporation Southern Liwa Area	-49	1840	-9.72
Evaporation Coastal Sabkha	-25	1500	-6.08
Evaporation Sabkha Matti	-2.57	780	-1.20

Balance Term	Quantity [10^3 m^3/d]	Section [km]	Mean Section Flow [m^2/y]
Outflow to the Sea	-8.9	207	-15.69
Inflow from the East	2.04	42	17.73

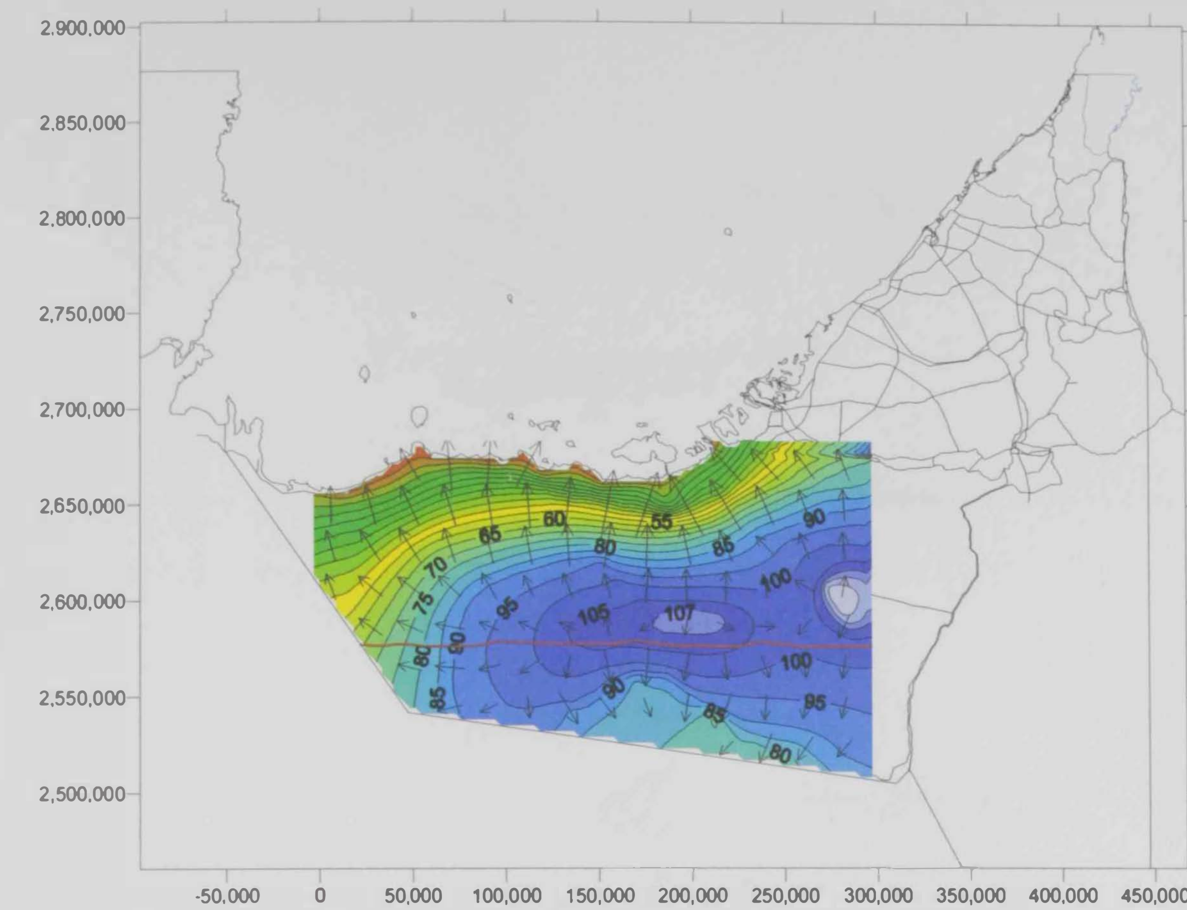

Figure 4.16 Natural groundwater flow vectors and the hydraulic divide

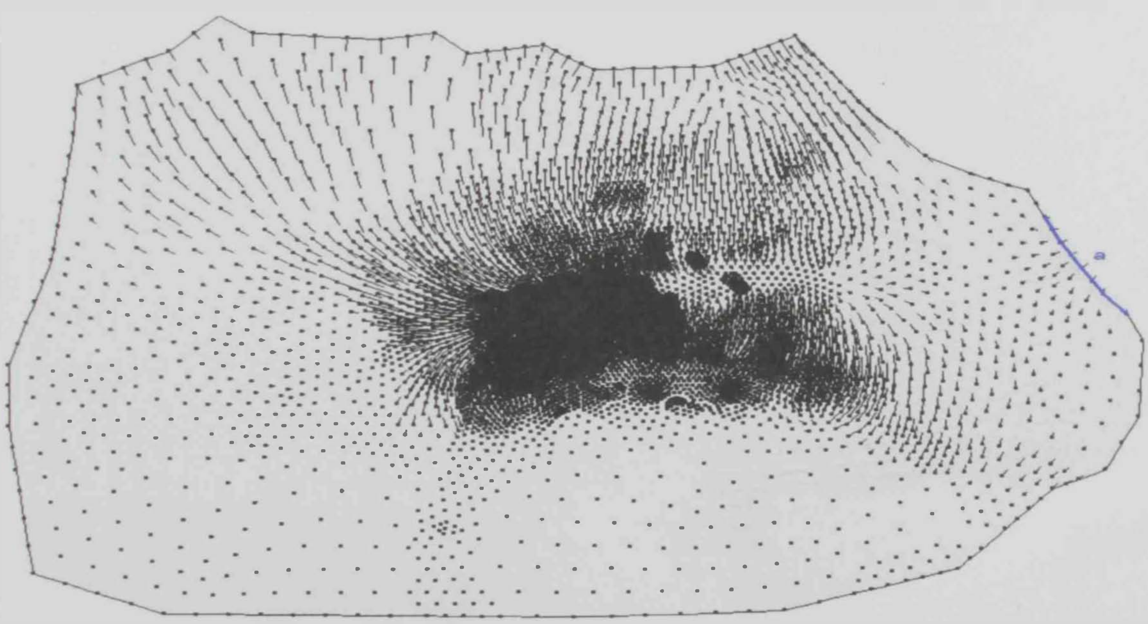

Figure 4.17 Calculated groundwater flow directions under the steady state conditions

Chapter V

Results & Analysis

5 Results and Analysis

The first part of this chapter shows an quick economical cost comparisons between surface water storage and aquifer recharge and recovery techniques. The second part will focus on the results of the feasibility of artificial recharge of groundwater resources in Abu Dhabi Emirate. The last part is devoted to the use of a numerical groundwater model (FEFLOW) to simulate and predict the fate and transport of contaminants that might be released from different sources in the vicinity of groundwater protection zones. Several scenarios, based on different assumptions, have been examined to study the potential impacts of contamination of groundwater near the infiltration basin (recharge) of the Liwa aquifer located in the Western Region of the Abu Dhabi Emirate. Through numerical modelling, the dimensions and location of the necessary groundwater protection zones are defined.

5.1 Economical Comparison between Water Storage Alternatives

This section presents a comparison between the available options for water storage, but is not intended to contain a thorough economic assessment of the full implementation of the project. The aim to provide the decision maker with a value showing the differentiation between the two options and making their judgment based on it.

Some assumptions were considered in this assessment:

- The project is of a strategic nature, which makes its implementation an obligation;
- The only possible alternative consists of storage in surface tanks; and
- The assessed formation at the selected site has a lateral extension with a high degree of homogeneity.

The comparison is made on analogous construction costs and do not include any operating expenses for both systems. The assumptions taken in the full implementation of both scenarios are:

- Total storage capacity is 3.6 BIG
- Desalinated water cost is 25 Dhs/1000 imperial gallons
- Surface facilities engineering and construction and water treatment plant cost estimates are included
- Permitting, Environmental Impact Assessment and infrastructure (roads, accommodations, cars, power lines, telephone lines…) cost estimates are included.
- Real Time monitoring system is considered but with a different number of units in the case of an ASR or Surface storage schemes.
- Allowance for contingency Specific assumptions for the Surface Storage scheme
- Volume for each storage tank is 20 MIG
- 180 tanks will be needed to store 3.6 BIG
- The cost of building a tank is 10 MDhs that includes soil assessment, and construction of the tank and foundation

Specific assumptions for the ASR scheme:

- Maximum recovery rate is 20 MIG for a continuous period of 180 days
- A total of 75 wells will be drilled: 45 injectors/producers and 30 monitoring wells
- Provisions are taken for surface and downhole data acquisition
- The cost of the water used to reach the required efficiency in the ASR is included

Table 5.1 provides some details about the specific costs that were considered for this comparison.

Table 5.1 Breakdown of cost comparison between ASR and surface storage.

Parameters	ASR	Surface Tanks
	Cost ($)	
Surface piping & Treatment	19,000,000	19,000,000
Cost of land (30 cent/m^2)	360,000	360,000
Infrastructure	1,300,000	5,500,000
Environment & permitting	60,000	60,000
Data Acquisition	850,000	N/A
Sub- surface Engineering	2,000,000	N/A
Drilling	9,200,000	N/A
Surface tanks construction	N/A	491,800,000
lost water	20,000,000	N/A
Real time monitoring system	750,000	1,800,000
10 % management fess	5,350,000	51,850,000
25 % contingency fees	14,700,000	142,600,000
Total	73,570,000	712,970,000

The result of the comparison is showing that the ASR cost represents 10% of the surface storage cost for a 3.6 BIG storage capacity. In dollar terms, this would be 73 million USD against approximately 713 million USD.

5.2 Feasibility of Artificial Recharge

In order to study the feasibility of artificial recharge in the study area, some steps were considered to achieve the goal. These steps are summarized hereafter.

The model was employed under the steady state conditions first step to ensure its functionality. Figure 5.1 shows the initial groundwater levels that were used in the model. View of the study area at steady state conditions is represented in Figure 5.2. Groundwater velocity isoline is shown in Figure 5.3 .

The resulted groundwater levels from the steady state conditions were used as initial groundwater levels for the transient conditions without any injection or abstraction for a period of about two years. This ensures that the model runs in a proper manner before considering the recharge/abstraction scenario.

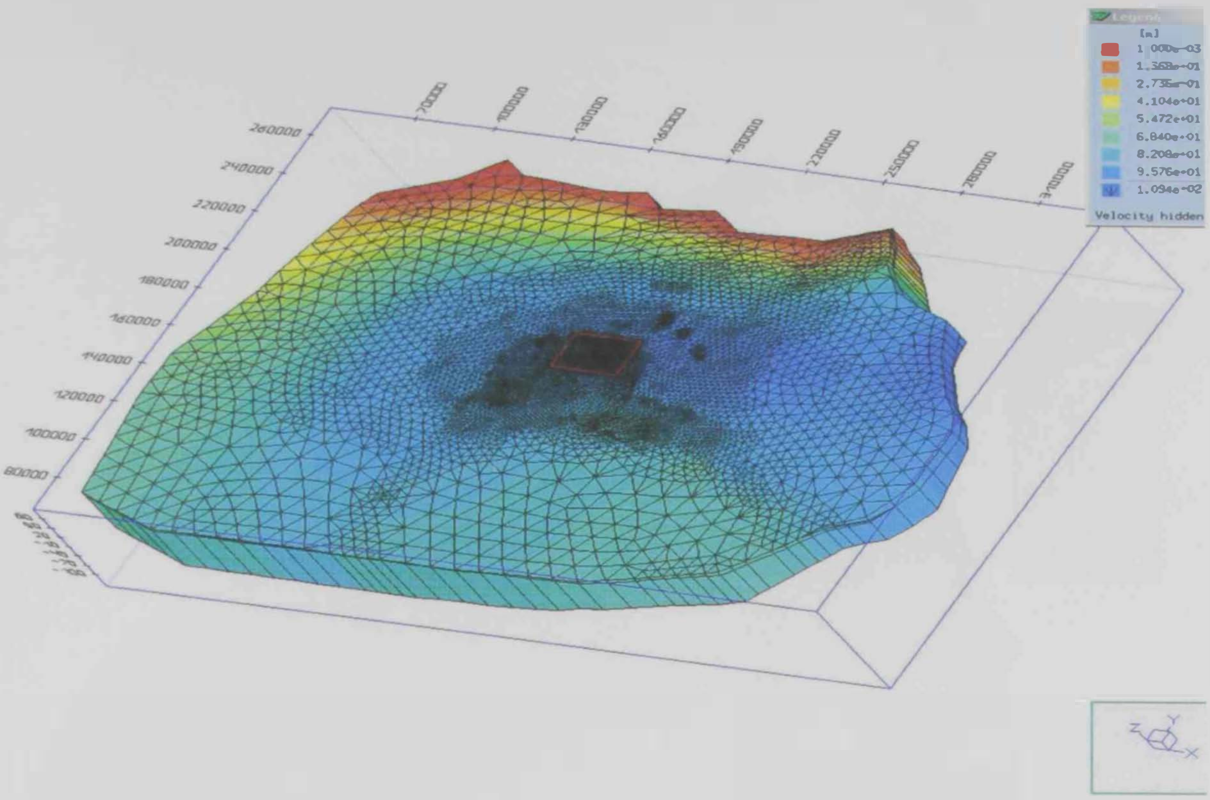

Figure 5.1 Initial groundwater table – steady state conditions

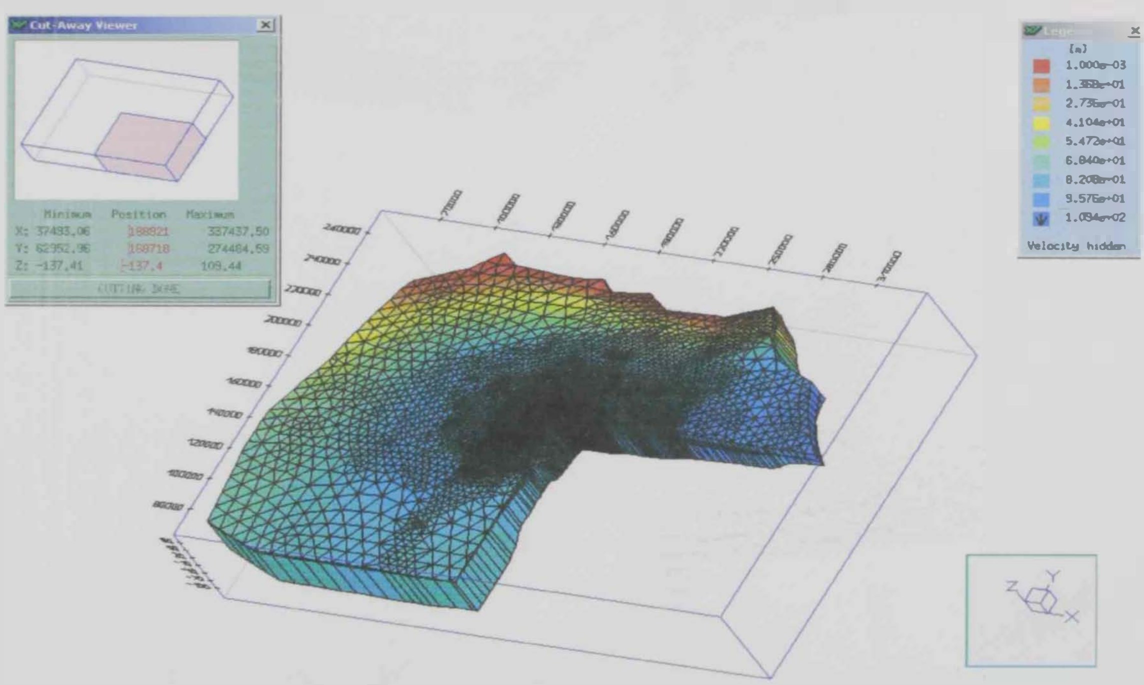

Figure 5.2 Cut view of the final groundwater table – steady state conditions

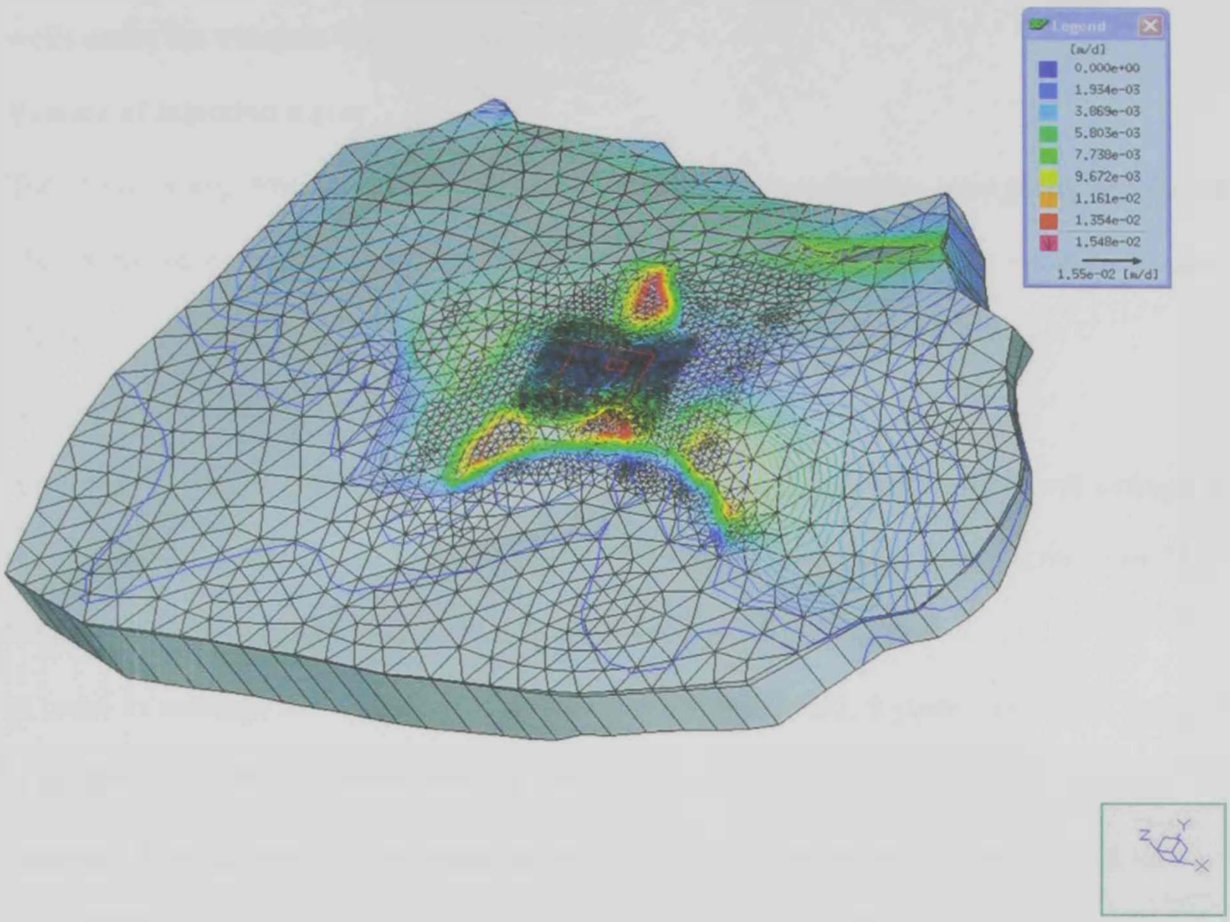

Figure 5.3 Final velocity isolines – steady state conditions

5.2.1 Design and analysis of well fields

The design of an artificial recharge system is generally based on the trial and error approach with the simulation model [Vidannarchchi et al., 1998]. During this design process, the arrangement of the well fields, the positions of the wells and the pumping rates from the wells under the transient conditions are selected.

Volume of injection water

The study is implemented to cover a domestic water demand under emergency conditions. The estimated per capita consumption of domestic water in Abu Dhabi is equal to 77 g/c/d [ADWEC, 2004].

Recharge process

Assuming a supply water requirement of three months is considered; the total volume of water needed is estimated to be 11,400 MG (43.15 Mm^3) with a recovery efficiency of 70 %, the total amount to be injected is equal to 16,425MG (61.5 Mm^3).

In order to recharge the 16,425MG for domestic water demand, 5 years continuous recharge is needed. Available recharge water is varied from year to another as shown in Table 5.2 Number of wells, duration and pumping rate during recharge process. Schematic diagram of the well fields for the recharge and recovery process is shown in Figure 5.4.

Recovery process

In order to simulate the recovery process, the model runs for 90 days after the 5 years of recharge.

Injection rate

The rate of injection mainly depends on the aquifer transmisivity, well hydraulics, clogging rate, and the availability of injection water [Mukhopadhyay et al., 1998]. The maximum recharge rate per well in the study area is estimated as 1,200 m^3/d (50 m^3/h) and for the recovery, 2200 m^3/d (91.7 m^3/h) [GTZ-DCO/ADNOC, 2002]. Based on this information and to avoid the excessive head buildup in the wells, the injection rates was restricted to be in the range of 650 – 800 m^3/d (27 m^3/h -33.5 m^3/h) for the recharge purpose and of 1,700 m^3/d (70.8 m^3/h) for the recovery process.

Number of wells

- *Recharge process*

The number of wells for the recharge purpose was varied from year to another depending on the availability of the desalinated recharge water as shown in Figure 5.5. Table 5.2 gives the number of wells with their corresponding pumping rates in m^3/d.

A total number of 69 wells will be needed at the end of the injection period for the whole five years. For the first year, 35 wells are operated with a pumping rate varying between 650 to 800 m^3/d to give a total injection rate of 6 MGD (22,712 m^3/d).

In the second year a total of 41 injection wells, 35 from the first year and only 6 new wells, are need with a pumping rate in the range of 600- 800 m^3/d. The available desalinated water for the injection in the third year is estimated to be equal to 10 MGD (37,845 m^3/d). Therefore, 50 wells should be operated with 9 new wells having an injection rate varying between 600- 750 m^3/d and the rest of the wells will be running at the same rate from the previous year.

In the forth year the recharge will take place through 52 wells with injection rates between 600- 800 m^3/d while in the fifth year 69 wells will be operated with an injection rate between 600- 800 m^3/d.

Table 5.2 Number of wells, duration and pumping rate during recharge process.

No of wells	Duration [years]	[days]	Pumping rate $[m^3/d]$
35 wells	1st year	0	650
		365	650
	2nd year	366	650
		731	650
	3rd year	732	750
Recharge		1097	750
	4th year	1098	800
		1463	800
	5th year	1464	600
		1829	600
6 wells	2nd year	366	650
		731	650
	3rd year	732	750
		1097	750
Recharge	4th year	1098	800
		1463	800
	5th year	1464	600
		1829	600
9wells	3rd year	732	750
		1097	750
	4th year	1098	800
Recharge		1463	800
	5th year	1464	600
		1829	600
2wells	4th year	1098	800
		1463	800
Recharge	5th year	1464	600
		1829	600
17wells	5th year	1464	600
Recharge		1829	600

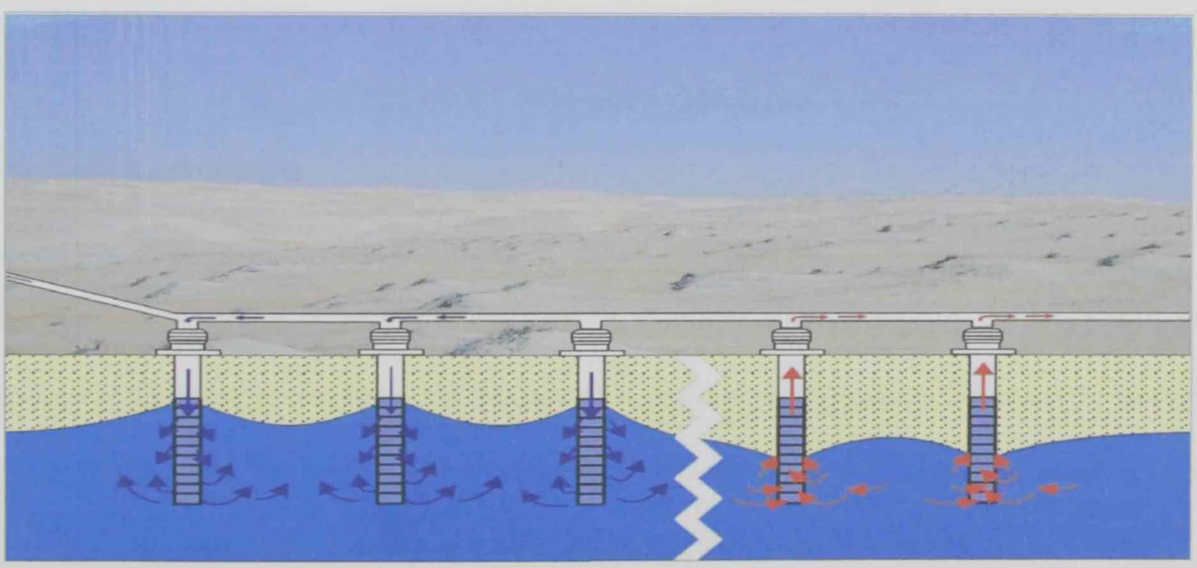

Figure 5.4 Schematic diagram of the well fields for recharge /recovery process

- *Recovery process*

For the recovery of the recharged water the whole 69 wells will be used with a relatively high pumping rate of 1,700 m^3/d (70.8 m^3h) running for 90 days.

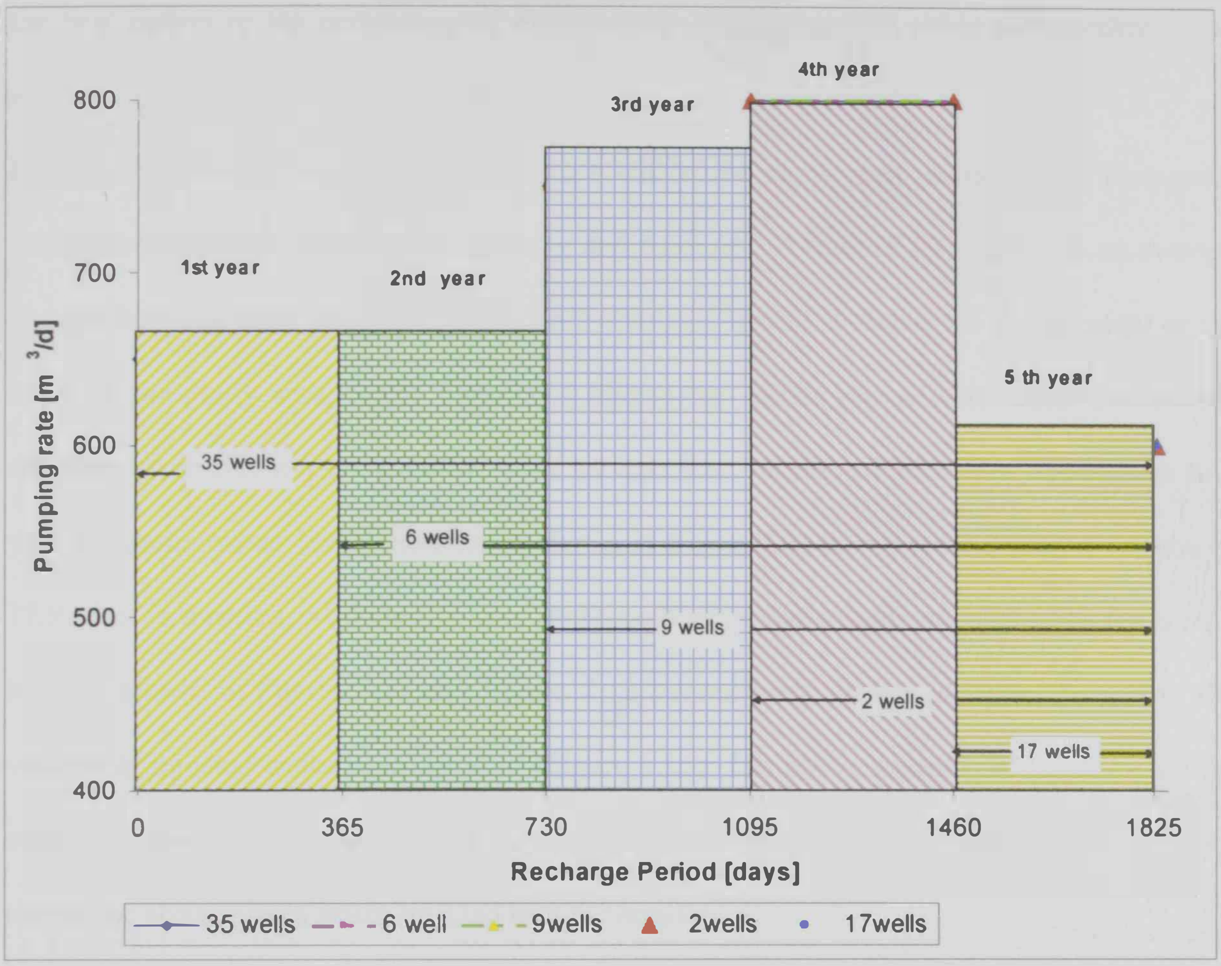

Figure 5.5 Wells with pumping rates

Arrangement of wells

FEFLOW was used to investigate the optimum arrangement of wells that would give the minimum head buildup. At the same time, a compact arrangement of the wells will also minimize the cost of pipelines for the injection and recovery process. Several arrangements were studied and the hexagonal arrangement was the . since it is mostly used in many artificial recharge sites around the globe and is recommended by experts in this fields. It is often used in situations where low water quality groundwater has to be replaced by infiltrated fresh water [GTZ-DCO/ADNOC, 2002]. The benefits from such an arrangement could be also viewed from the high rise of groundwater table and smaller area of hydraulic impact, so the protection zone will be relatively limited. Uniform drawdown below the well fields is also one of the advantages of using such arrangement. In addition, the hexagonal arrangement was the best option in the percentage of the recovery as compared to other arrangements (line wells for example).

Figure 5.6 shows an over view map with a cut of the study area including the hexagonal arrangement of the wells used for recharge and recovery. A total of 69 wells with an average spacing between them less than 200 m were chosen. Figure 5.7 shows the arrangement of the wells in the study area, while Figure 5.8 shows the initial groundwater lever before any recharge or abstraction takes place. After an injection volume of 16.5 Mm^3 during the first year the groundwater levels increase as shown in Figure 5.9. In the second year, a volume of 17.9 Mm^3 is injected, while in the third and fourth years the volume of injected water is 18.9 Mm^3 as shown in Figure 5.11 and Figure 5.12, respectively. In the last year, year five, the volume of injected water is 15.25 Mm^3 (Figure 5.13). The above amounts of injection are for each year separately and are not the accumulated injected water through the years. Table 5.3 shows the accumulated water injected into the aquifer.

Table 5.3 Accumulated water injected into the aquifer (from 0 days to 1829 days)

Year	Accumulated water [Mm^3]
First	16.5
Second	33
Third	51.06
Forth	69.9
Fifth	82.82

Figure 5.14 shows the head distribution as resulted from the simulation. The average natural groundwater level at the study area is 107 m. After five years of recharge the groundwater level rise by about 28 m. Wells at the centre were the most affected by the recharge more than the ones at the edges. Wells at the corner have an average groundwater level rise of about 10 m. As shown from previous figures, the effected area with the recharge process is within the study area and the area surrounding the wells has not been very much affected. The hydraulic impacts on groundwater levels within the study area from one year to another are shown in Figure 5.15- 5.20, respectively.

Neither the rise of water table nor the drawdown exceeded 4m in the surrounding area of the well fields as shown in Figure 5.21- 5.25, respectively. These Figures represent the difference between the initial groundwater level and the rise in groundwater level at the end of each recharge years. The average groundwater level rises at the end of each recharge year is 14, 20, 25, 27 and 28 m, respectively. The drawdown of water table is greater at the wells and decreases with increasing the distance from the wells as shown in Figure 5.26. The affected area of the recharge process is shown in Table 5.4.

Table 5.4 Impact area of recharge.

Groundwater level rise (m)	Impact area (km^2)
6	13
5	15
4	16.5
3	19
0.1	50

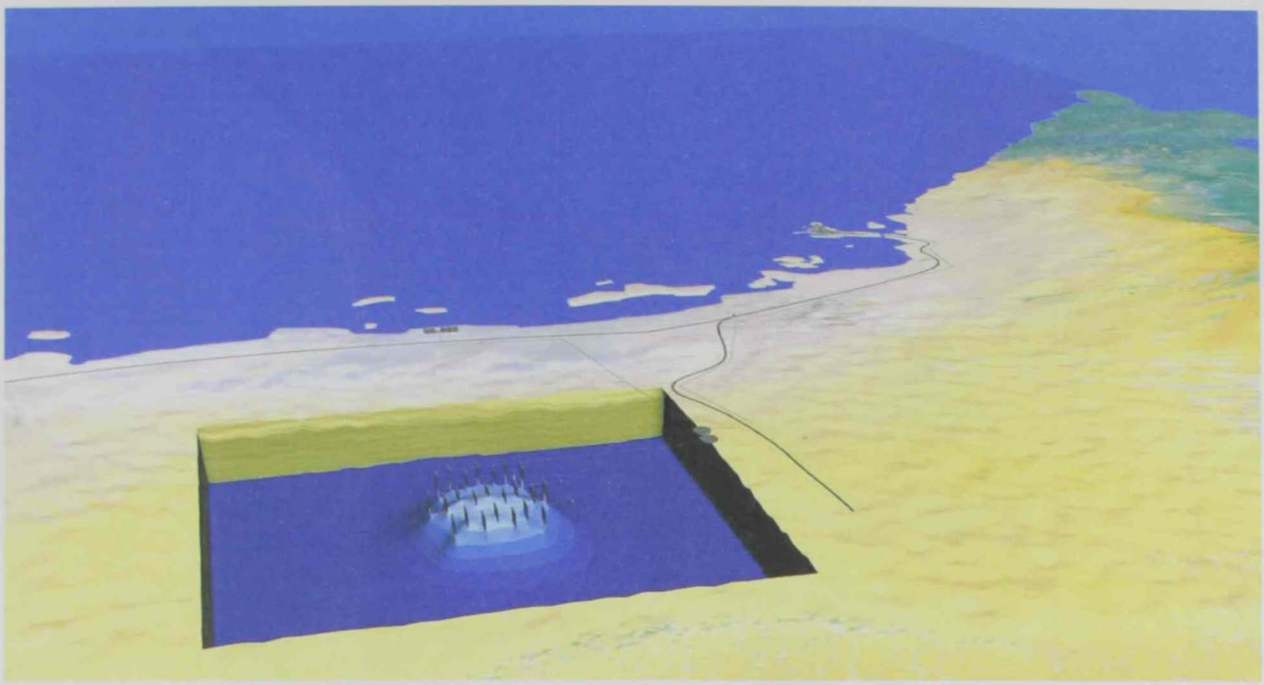

Figure 5.6 Schematic diagram of the overview map of the arrangement of the well in the study area

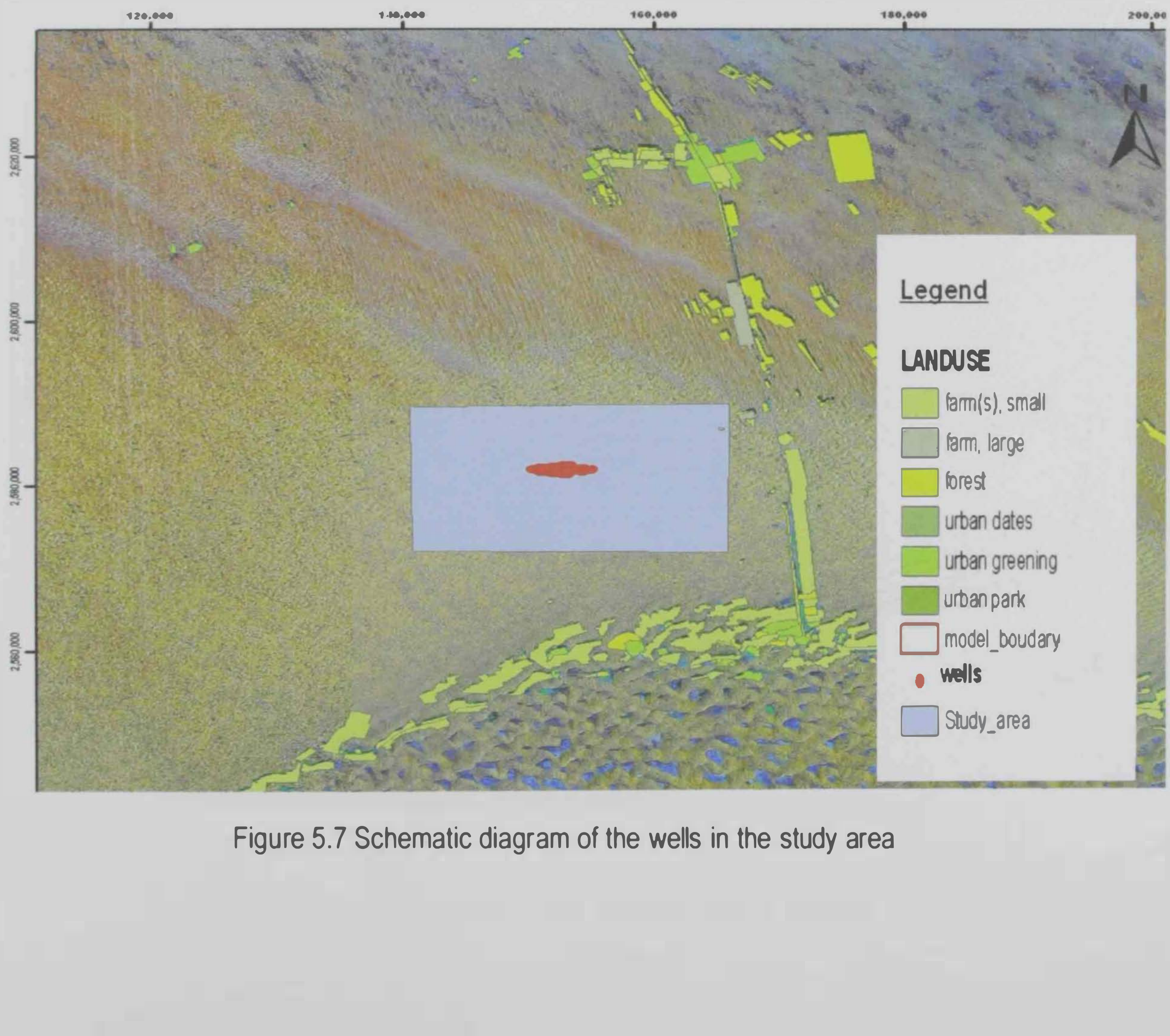

Figure 5.7 Schematic diagram of the wells in the study area

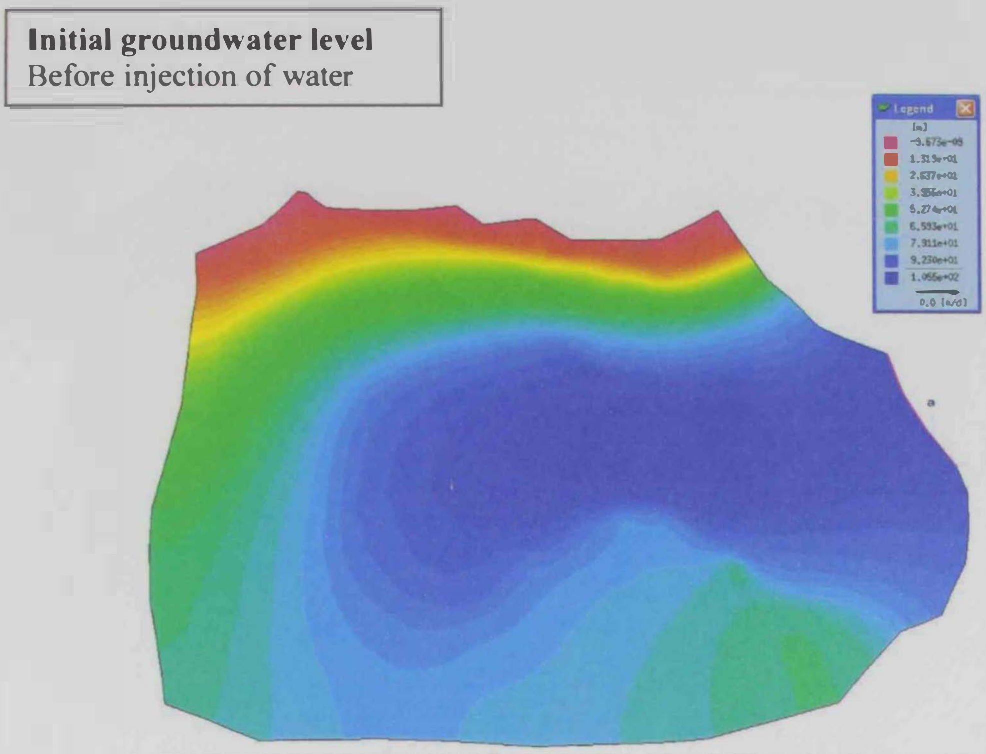

Figure 5.8 Initial groundwater level- study domain

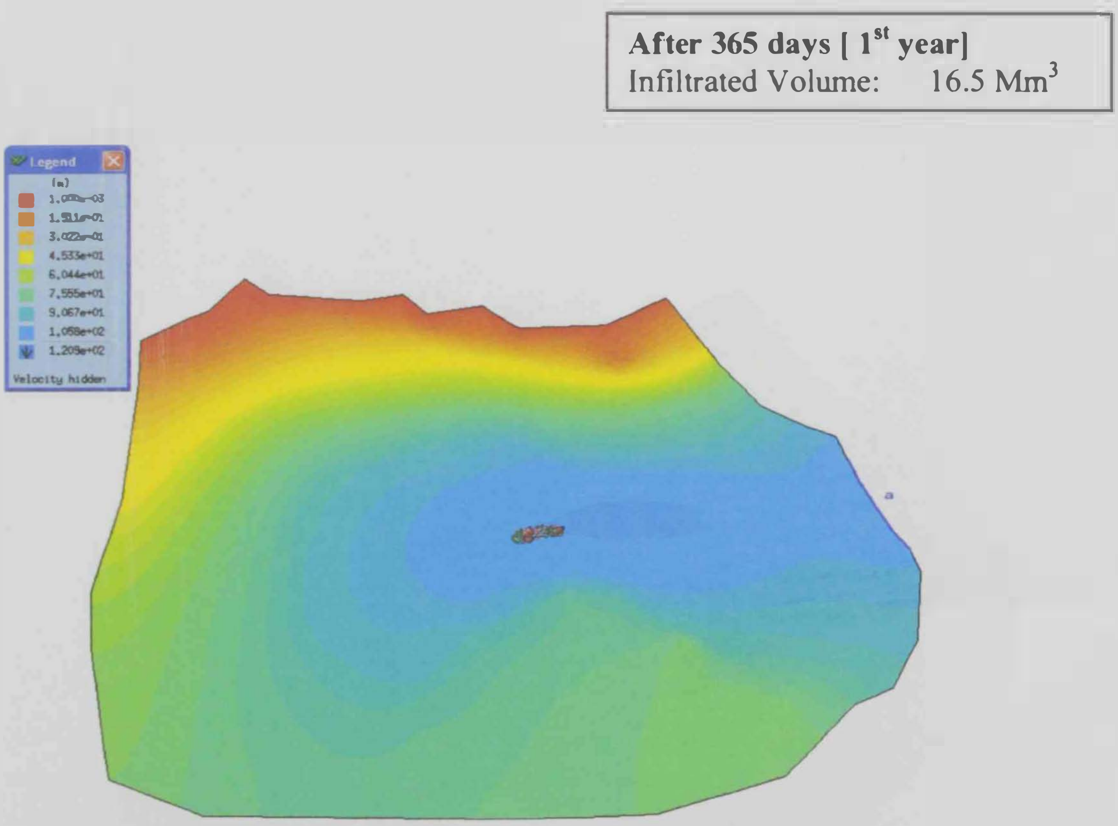

Figure 5.9 Groundwater level- after one year of injection

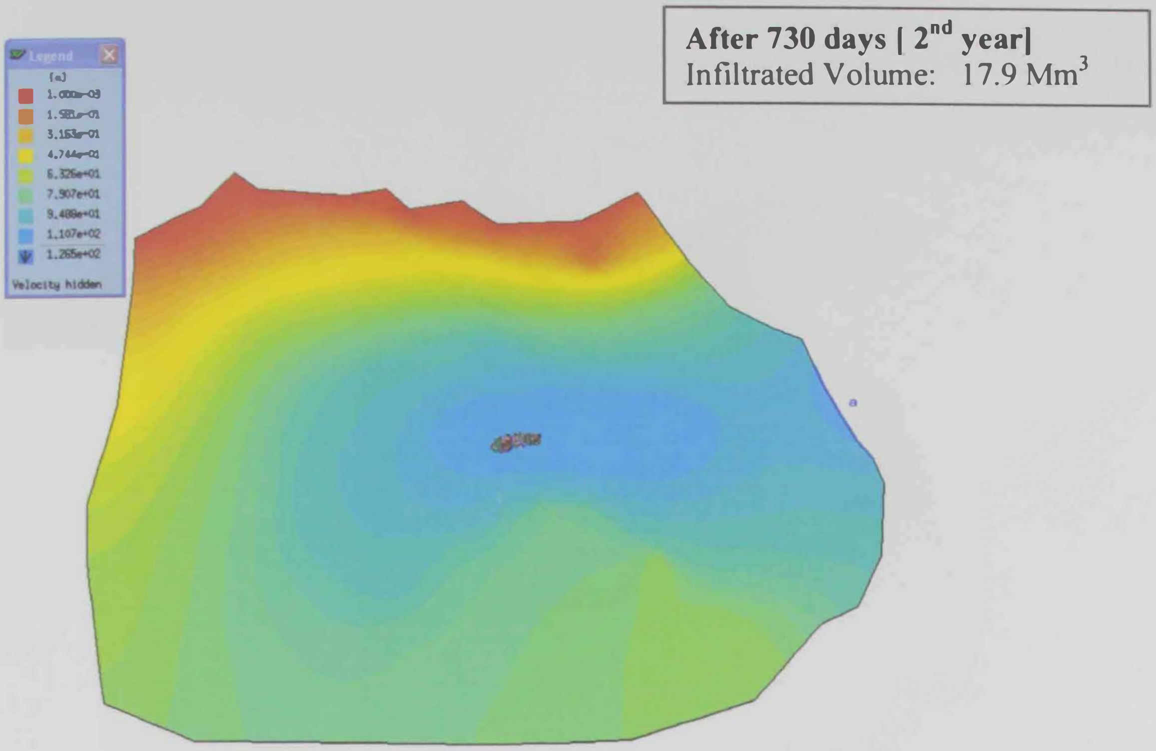

Figure 5.10 Groundwater level- after two year of injection

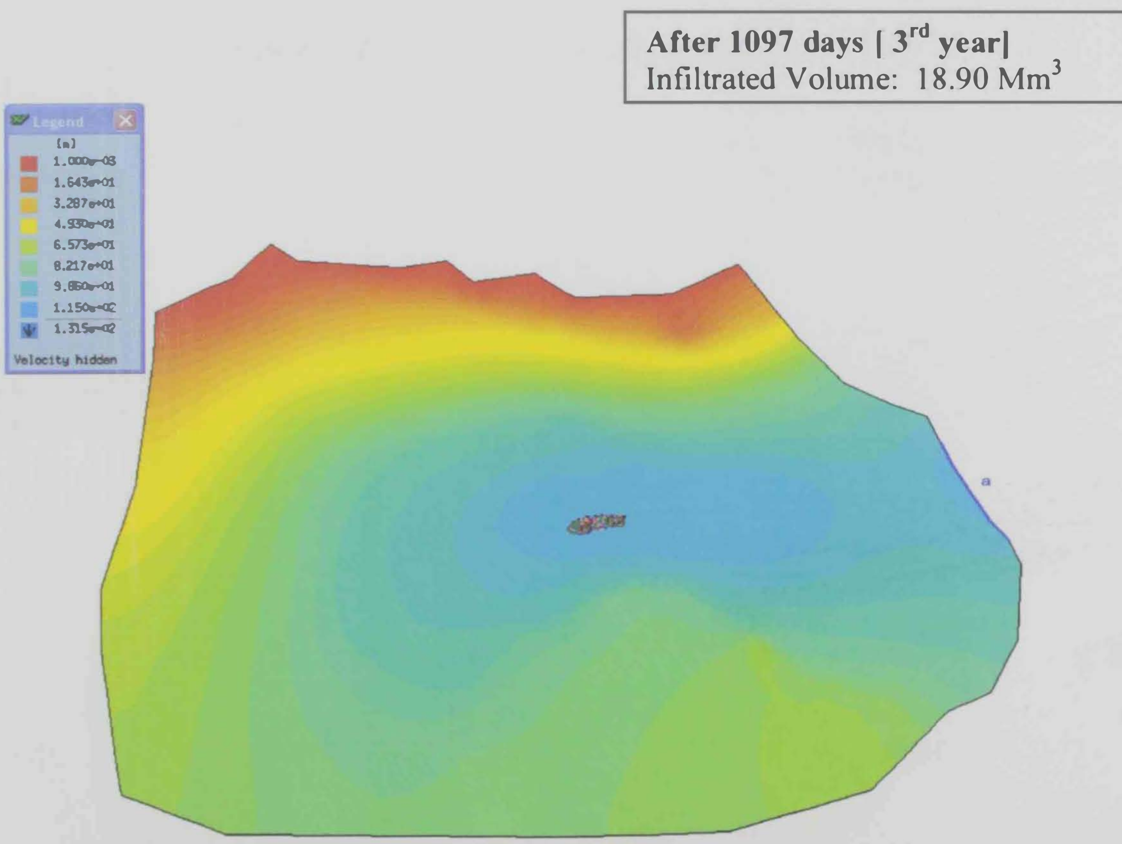

Figure 5.11 Groundwater level- after three year of injection

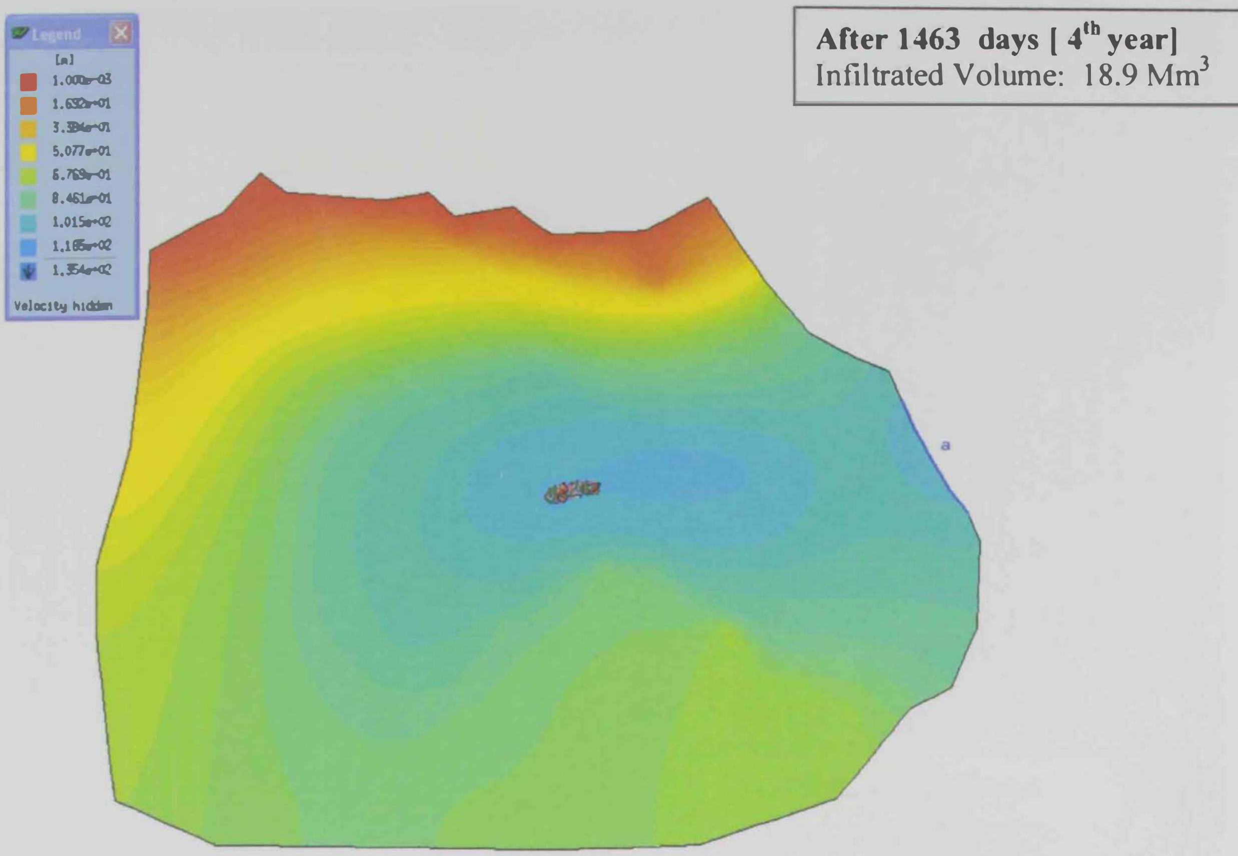

Figure 5.12 Groundwater level- after four years of injection

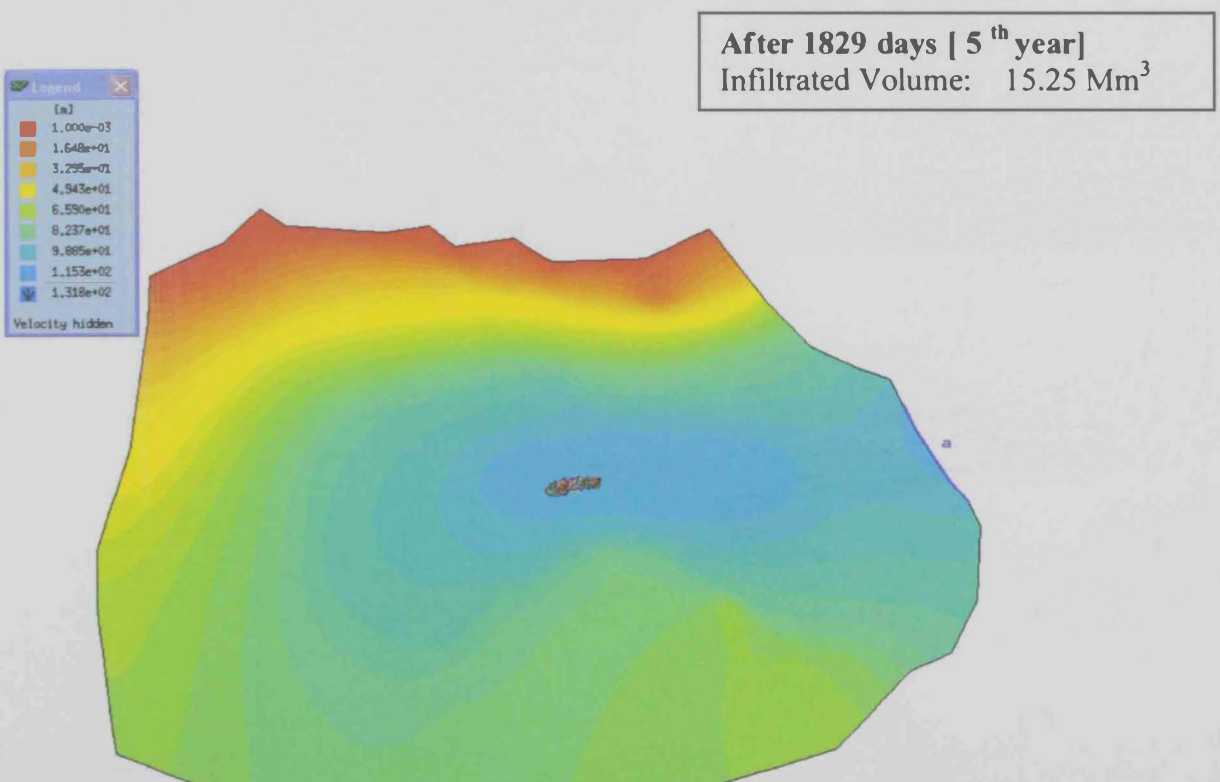

Figure 5.13 Groundwater level- after five years of injection

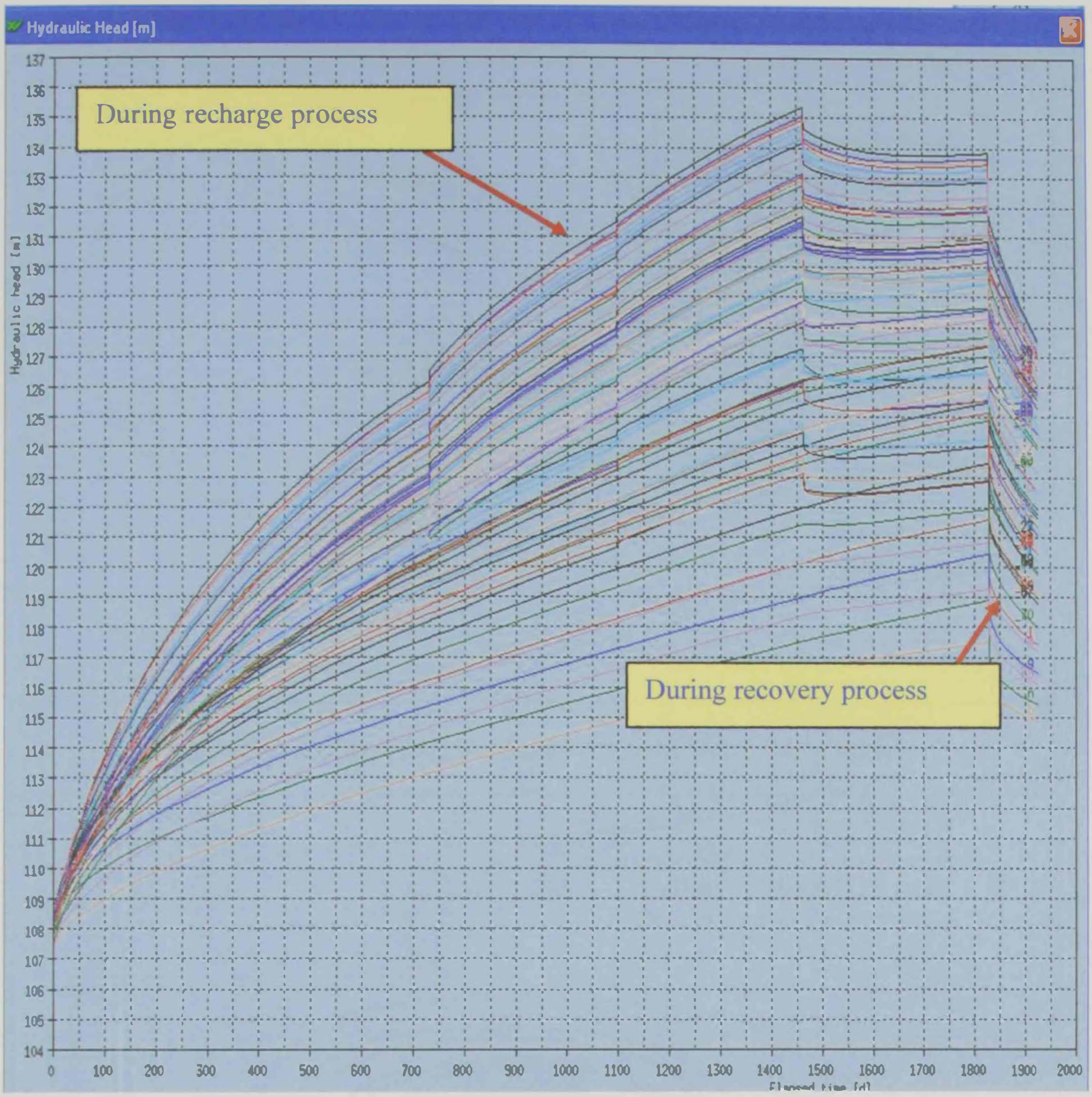

Figure 5.14 Head distribution for the recharge and recovery (m)

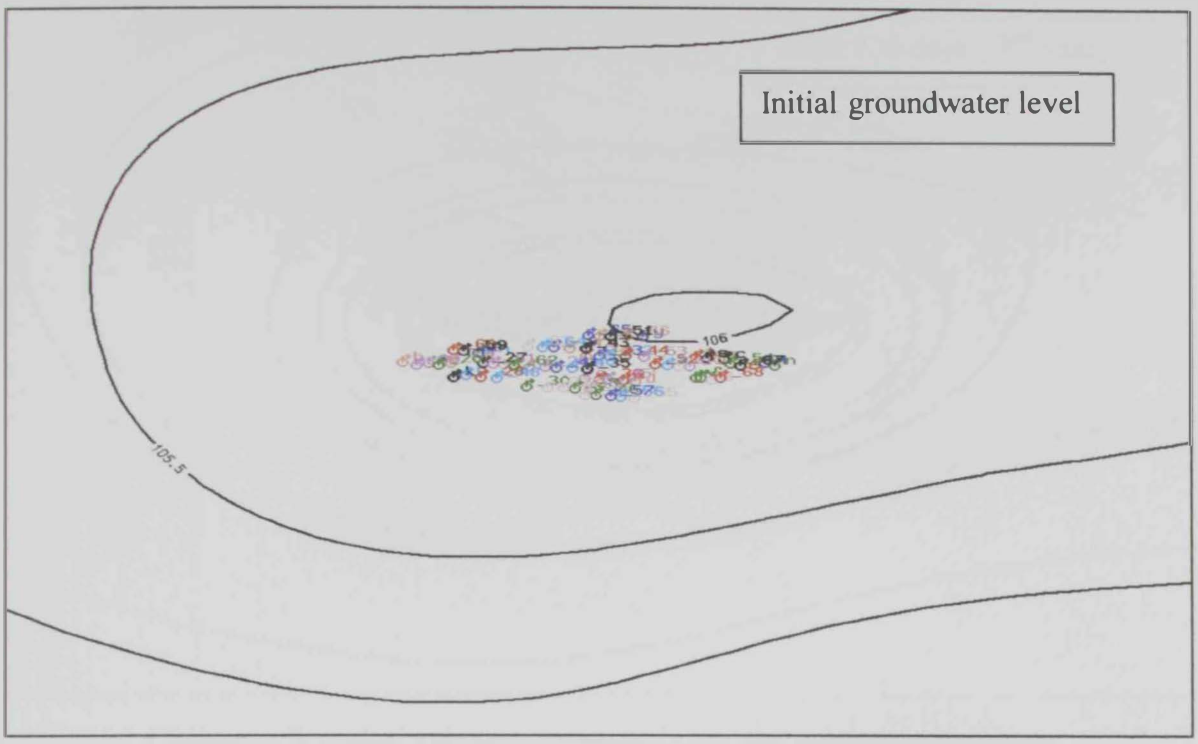

Figure 5.15 A contour map of groundwater levels before the recharge

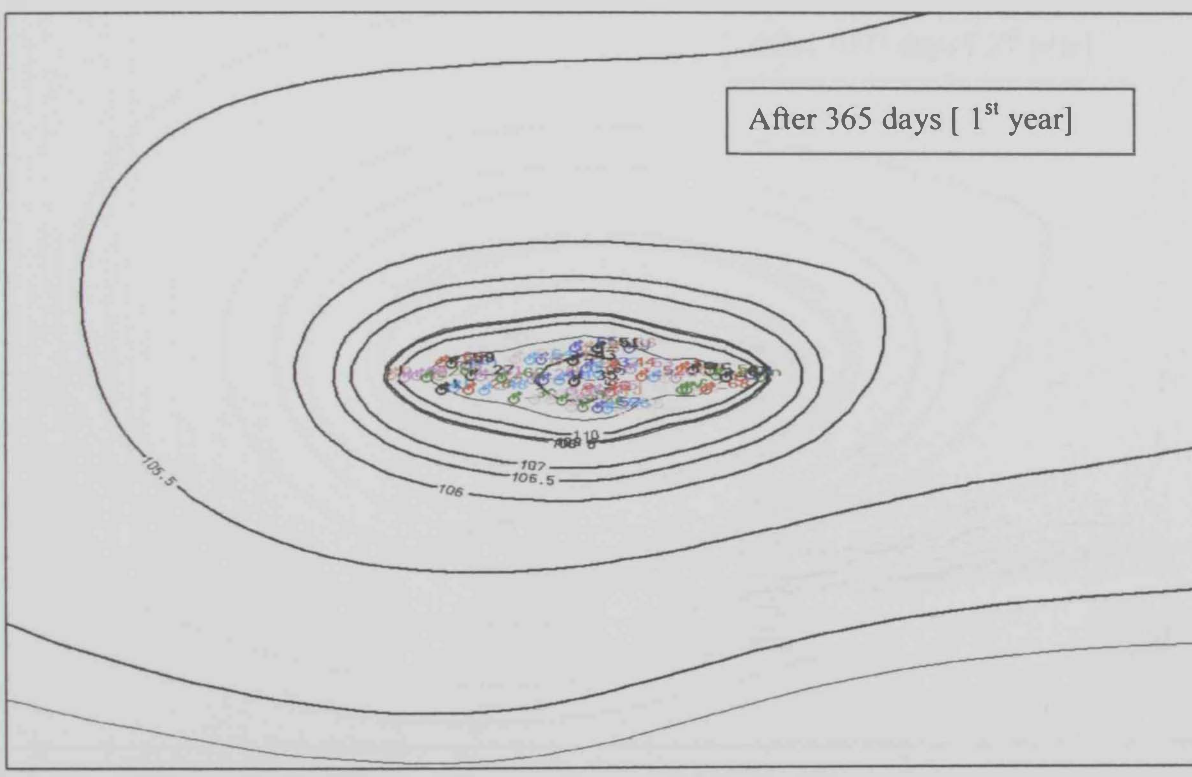

Figure 5.16 A contour map of groundwater levels after one year of recharge

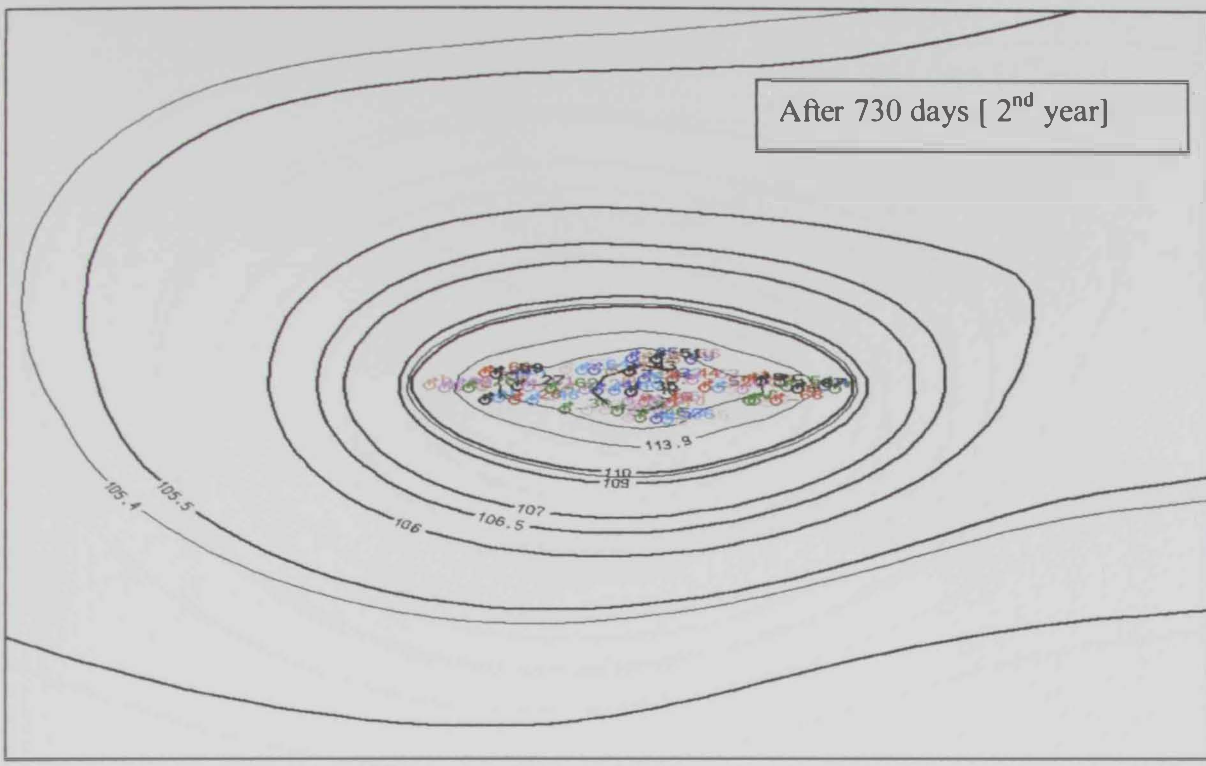

Figure 5.17 A contour map of groundwater levels after two years of recharge

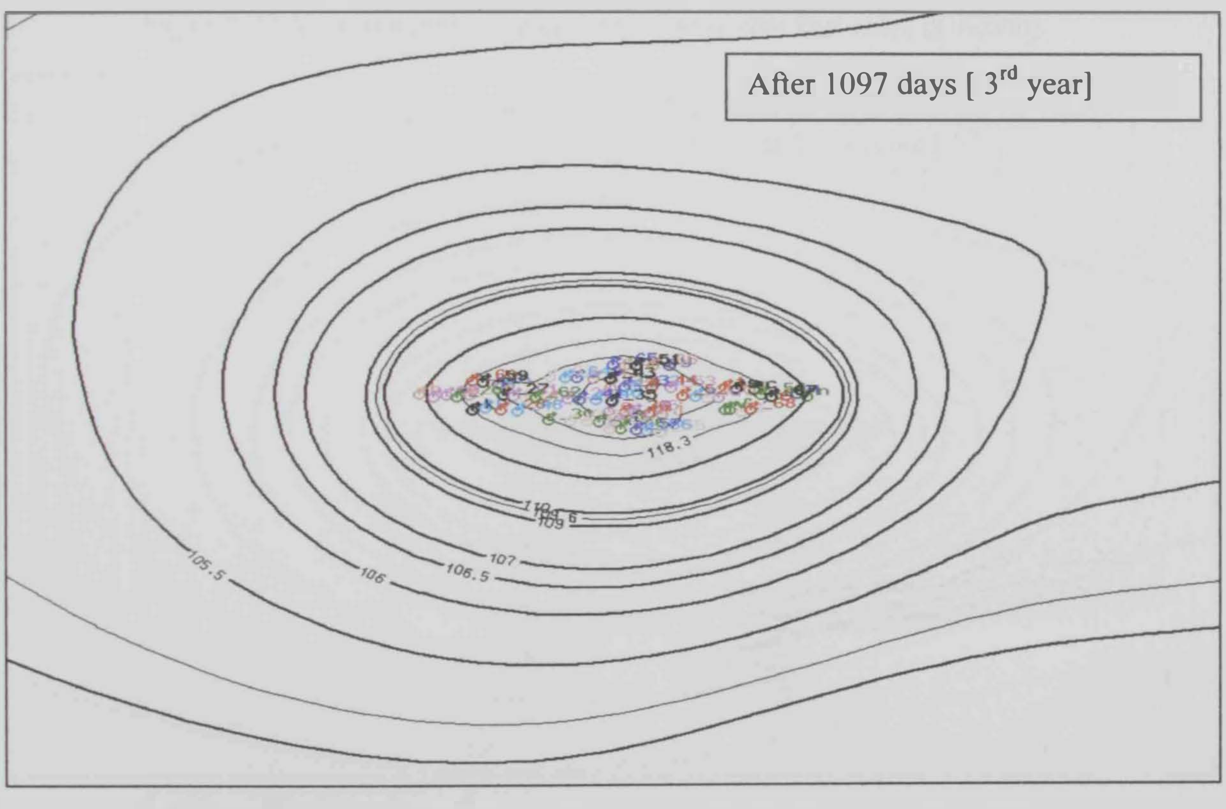

Figure 5.18 A contour map of groundwater levels after three years of recharge

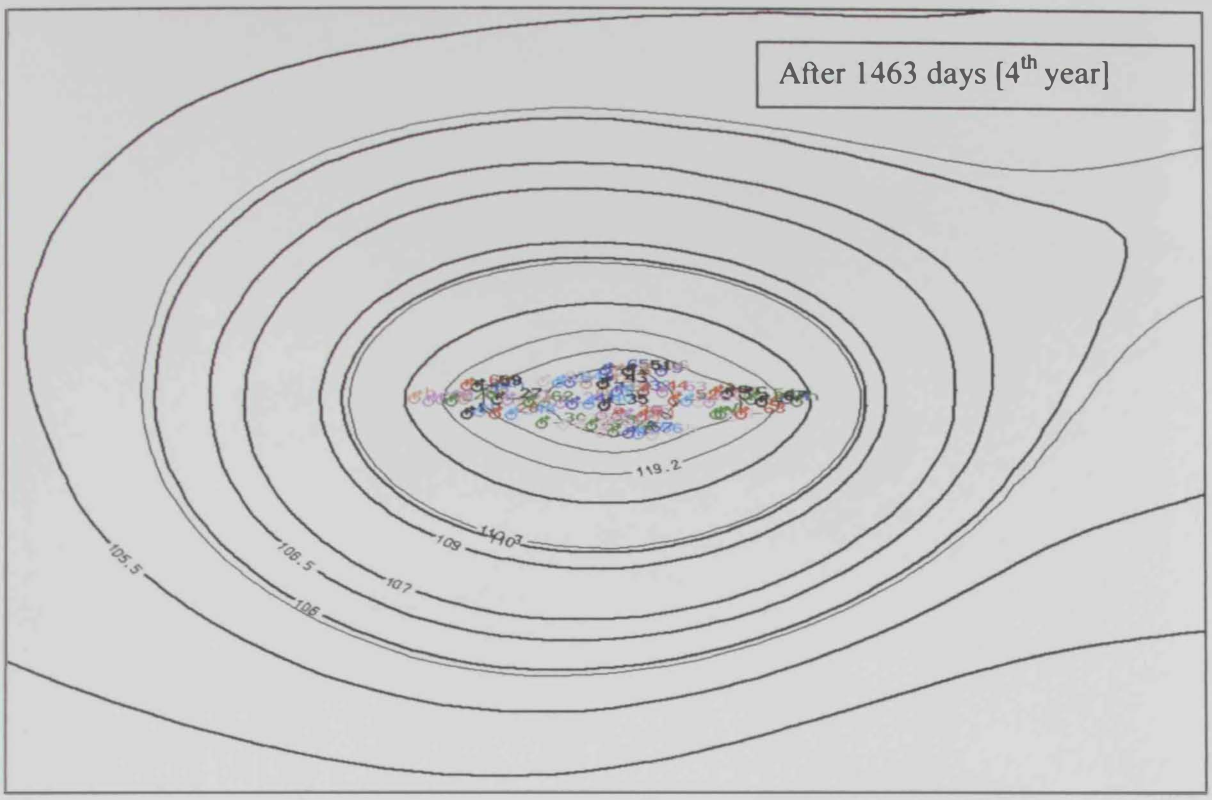

Figure 5.19 A contour map of groundwater levels after four years of recharge

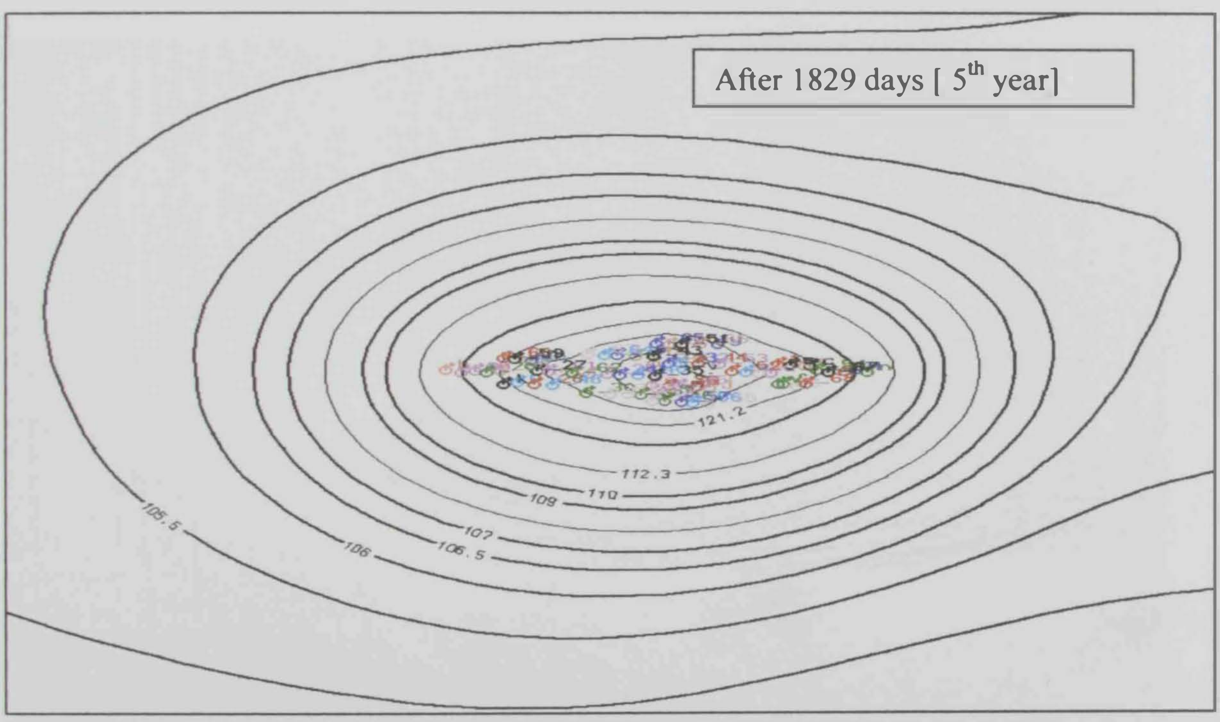

Figure 5.20 A contour map of groundwater levels after five years of recharge

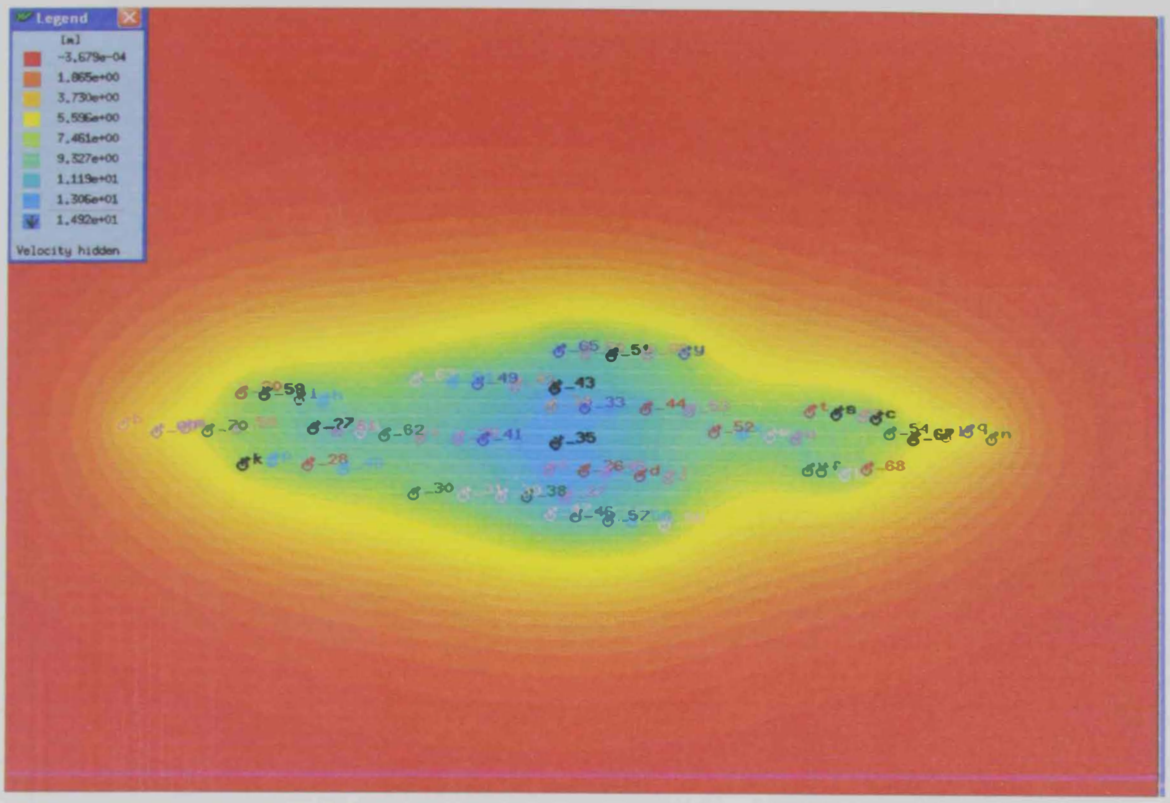

Figure 5.21 Difference between initial and final groundwater levels after 365 days of recharge

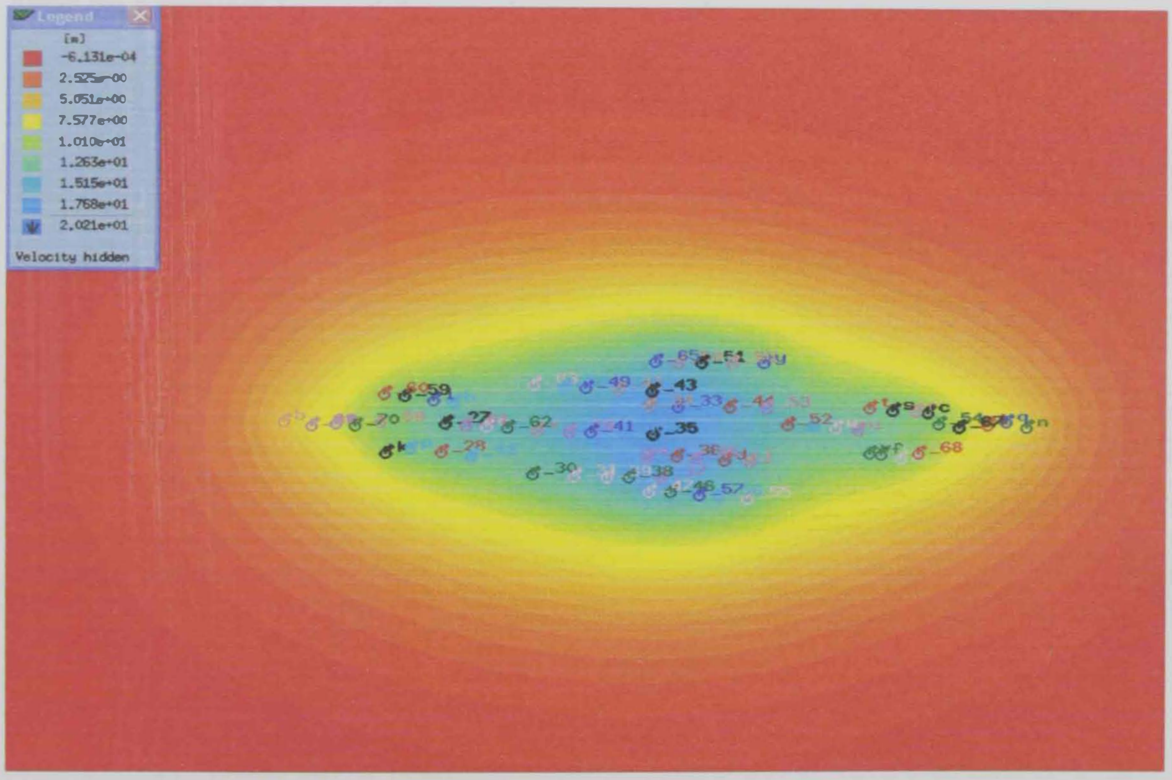

Figure 5.22 Difference between initial and final groundwater levels after 730 days of recharge

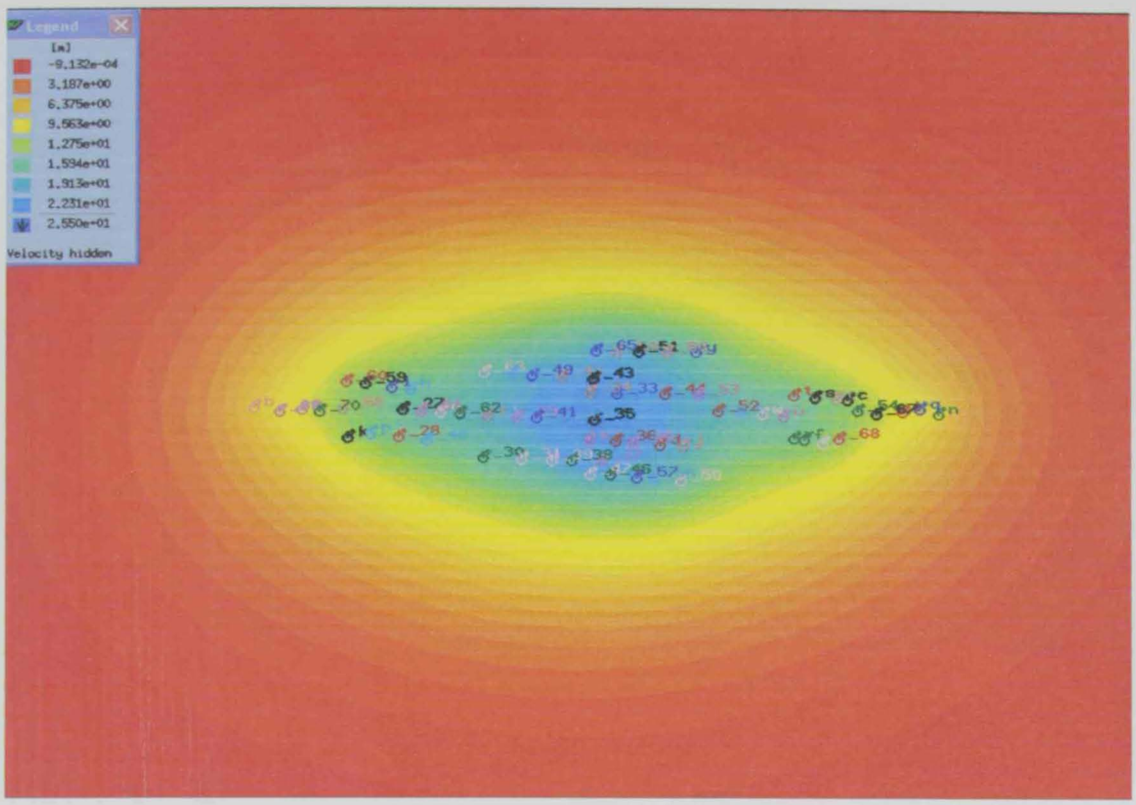

Figure 5.23 Difference between initial and final groundwater levels after 1097 days of recharge

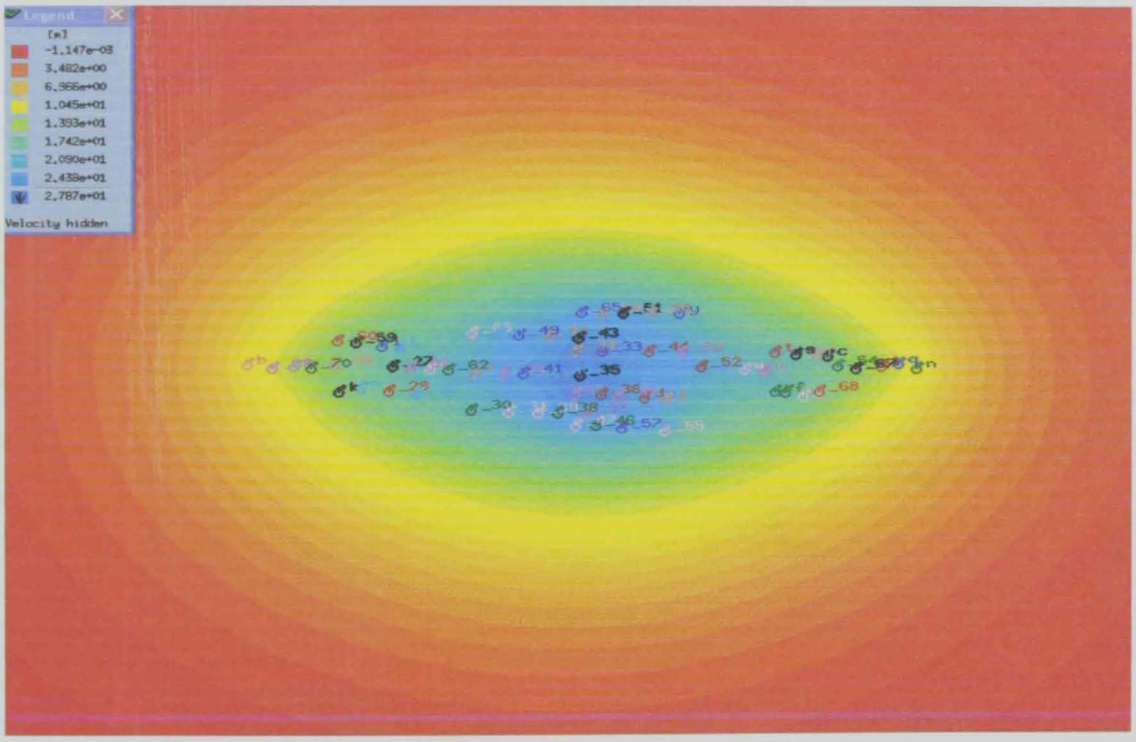

Figure 5.24 Difference between initial and final groundwater levels after 1464 days of recharge.

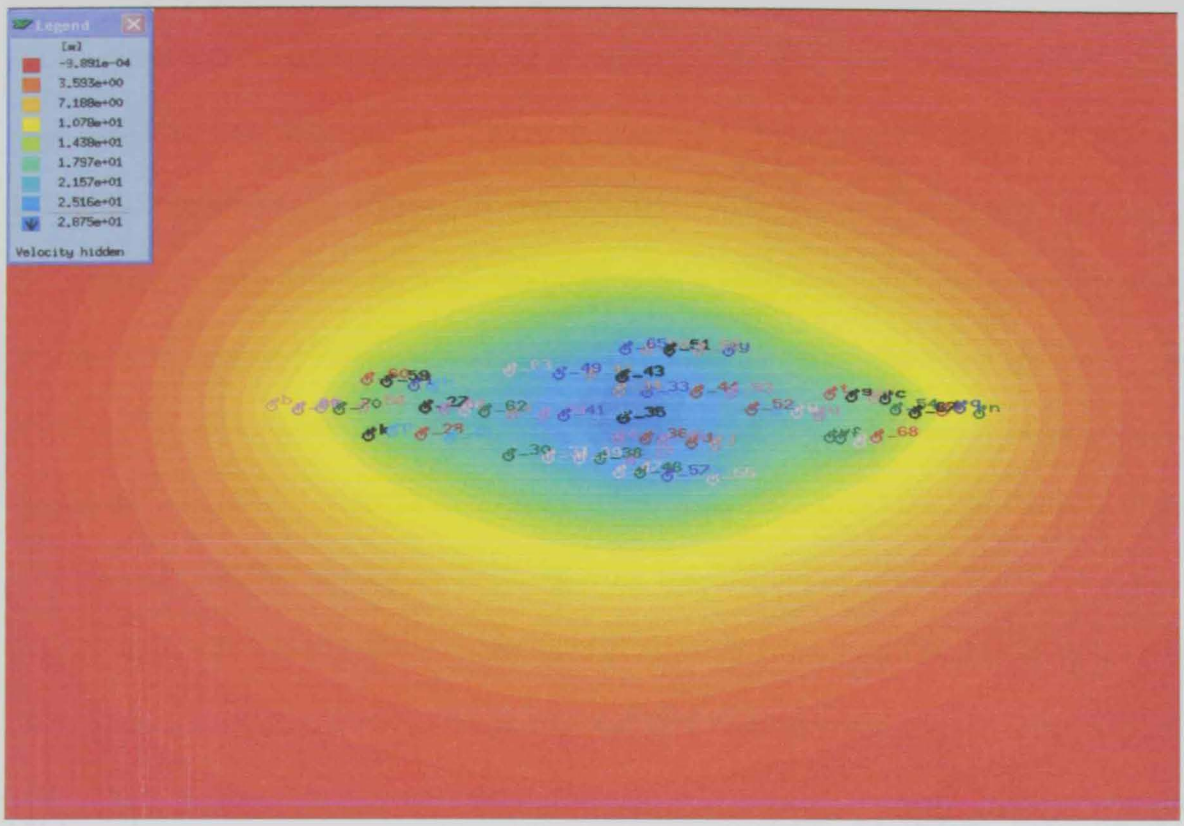

Figure 5.25 Difference between initial and final groundwater levels after 1829 days of recharge

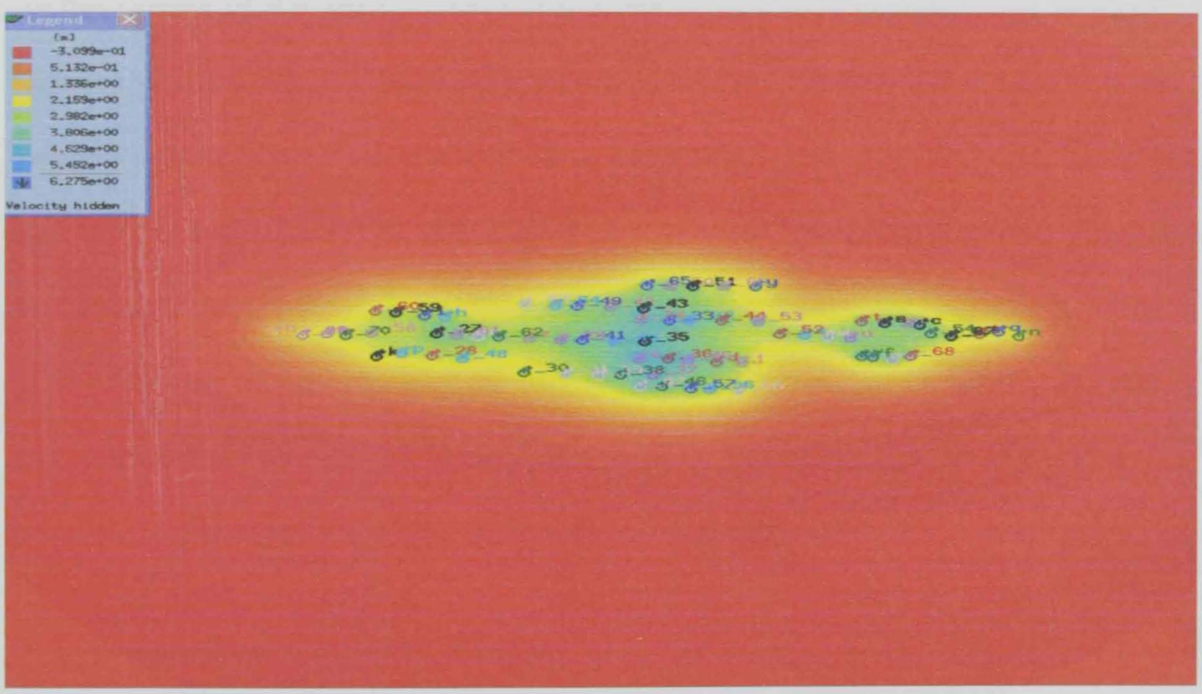

Figure 5.26 Drawdown of groundwater level after 90 days of abstraction

Percentage of the recovered water

To calculate the percentage (efficiency) of recovery, the first kind mass boundary condition was assigned with 180 mg/l to represent the injected desalinated water and the natural background groundwater in the study area was set to be in the range of 1,500 to 2,000 mg/l. The mass transport boundary condition consists of a fixed concentration boundary condition of 180, representing 100% desalinated seawater. This boundary condition has been assigned on all wells. Also, some constraints were applied which deactivate the boundary conditions during the recovery of water to be limited to less than 500 mg/l. Thus, the water is produced with its computed concentration resulting from the mixture of desalinated water and native groundwater, not with the fixed concentration. Therefore, the concentration was fixed at the 69 injections well during the recharge process. The average mass concentration from all the wells was considered to represent the percentage of recovery. Therefore, if the concentration was 180 mg/l then the percentage of recovery is 100 %.

As the screens of the wells do not reach to the aquifer bottom, a potential constraint was assigned ensuring that the well drawdown is limited to the depth of the lower end of the respective well screen. In addition, a rising water table must not get too close to the ground surface. Therefore, minimum and maximum constraints for the groundwater level at each well were assigned. The wells must be capable to recover as mush as possible of the water, without causing an upconning of the deeper groundwater with lower quality. Therefore, the maximum drawdown was set to be 12 m which is represents a water level of 92 m. The percentage of recovery was about 70 % with TDS less than 500 mg/l.

5.3 Possibility of Contamination

Groundwater, under most conditions, is safer and more reliable for use than surface water. The main reason for this is that surface water is more readily exposed to pollutants from factories, for example, than the groundwater. This does not mean that groundwater is invulnerable to contamination. Once groundwater is contaminated, it will be very costly to remove the contaminates. Chemicals that are easily soluble and may penetrate the soil are the prime candidates to cause groundwater pollutions.

The list of possible contaminates of groundwater is long. Some contaminates such as arsenic, may occur naturally in some areas of the earth's crust. Salt with high concentrations can be considered a contaminate. Groundwater on the average is more saline than surface water but still is not considered saltwater. Also, groundwater contains more sodium, boron, and nitrate than the surface water does. Groundwater may also contain high levels of sulphur, magnesium, and calcium.

A potential pollution problem can still reach groundwater wells miles away through underground water currents. For example, a chemical spill at an industrial plant distant from a residential area aquifer could infiltrate the ground and eventually enter the aquifer system that an entire community uses for their private wells. This situation could have devastating effects. Groundwater pollution is characterized by three distinctive properties:

- It is a very slow process.
- It is a stable process. Pollution remains in aquifer as a result of the slowness of groundwater flow.
- It is a local phenomenon. It corresponds to the source of pollution due to the slow migration of pollution.

Water with dissolved contaminants must travel through the unsaturated zone and arrive insufficient quantities at the water table to be of environmental concerns. This soluble chemicals move with the groundwater as it flow [Trautmann et al., 2005]. However, insoluble chemicals do not mix fully with groundwater, and their flow patterns depend on their densities relative to water. The rate of movement and degradation of the components depend on a variety of chemical, physical and biological processes.

5.3.1 Possible contamination sources in the study area

Groundwater moves very slowly. Therefore contaminates may not appear in a well until many years after entering the groundwater. For this reason a comprehensive assessment of the potential contamination sources that may affect the study area is established. A previous study conducted in Liwa showed that there are possible contaminations to the aquifer system specially from the oil fields [Woodwar, 1996]. Based on this, the potential contaminant sources are identified as follows:

1. Waste dumping and landfill sites.
2. Oil fields / Petrol stations.
3. Farms

Figure 5.27 shows the locations of possible groundwater contamination sources that may effect the artificial recharge site. The distance between the study area and the different sources is indicating in Table 5.5.

Table 5.5 Potential contamination sources

Source	Distance to study area [km]
Bu Hasa oil field	32
Northern Petrol station	36
Southern Petrol Station	About 25
Dumping area	16

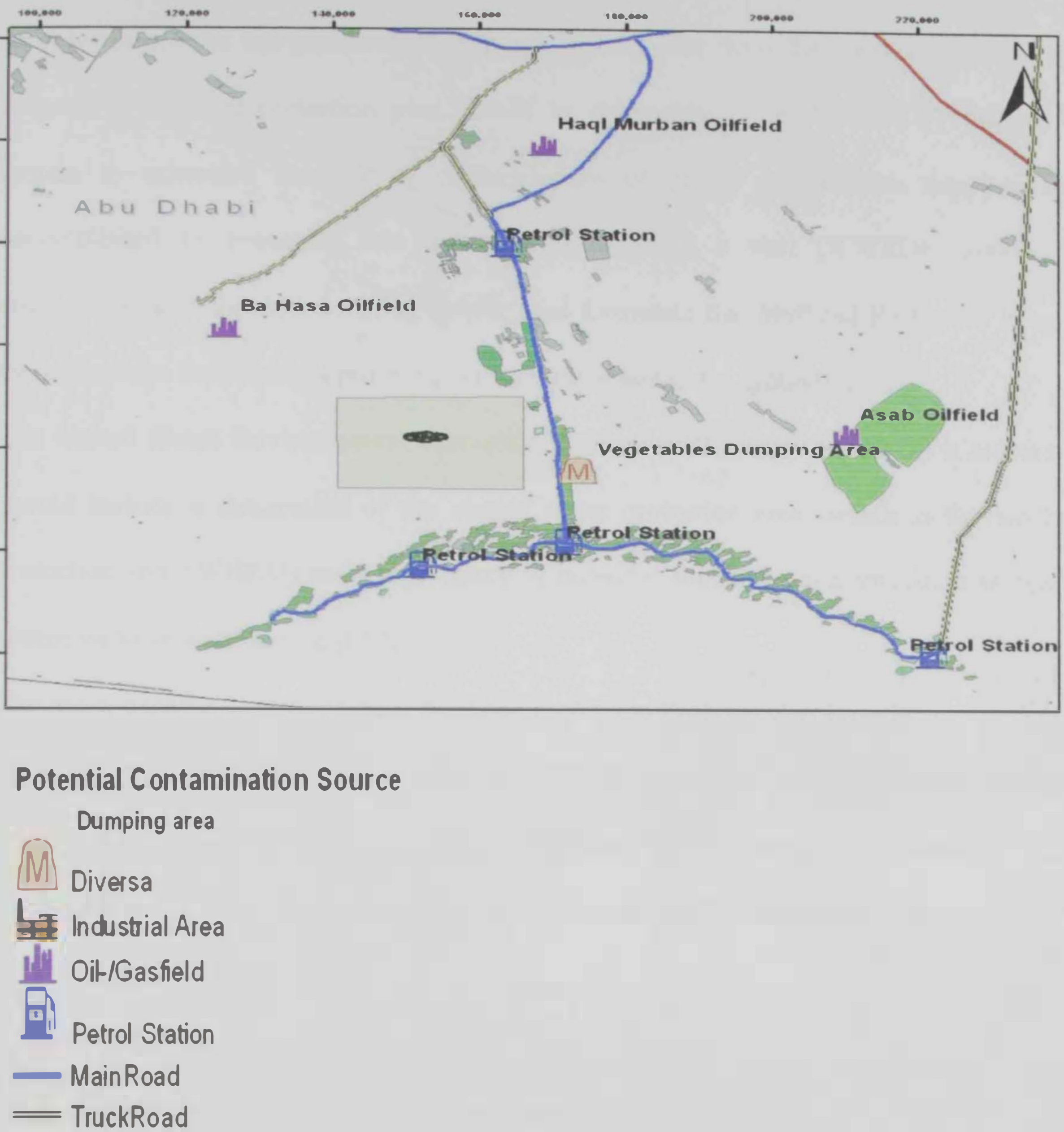

Figure 5.27 Potential contamination sources

Wellhead Protection

In order to prevent the groundwater recharge/abstract area from the possible contamination sources, a wellhead protection plan should be determined. The Wellhead Protection is a system to minimize the risk of contamination of public groundwater supplies. It is accomplished by managing the land area surrounding a well [WWRDS, 2006]. The environmentalist should be willing to plan and formulate the Wellhead Protection Program throughout the development process to ensure uncontaminated groundwater.

The United States Environmental Protection Agency (EPA) requires that each assessment should include a delineation of the source water protection area (which is the wellhead protection area [WHPA]) and an inventory of potential sources of contamination within the source water protection area [EPA, 1993].

The major benefits of developing a Wellhead Protection Program plan include:

1. Control and prevent contamination specifically within the local source protection area and to effectively implement cleanup measures if an accidental spill occurs;
2. Save resources by avoiding costs of cleaning up groundwater and/or providing an alternate water supply;
3. Reduce the potential health risks;
4. Reduce the potential environmental risks;
5. Protect a water supply for the future;
6. Develop a systematic approach to help safeguard the available water resources against the threat of contamination [EPA, 1993].

Steps to prepare the wellhead protection area

There are many methods that could be used to determine the area to be protected from different contamination sources. The steps below represent the important components in this protection process.

Step 1: Delineation

Delineation is defined as a process to identify a specific area that needs protection, the Wellhead Protection Area. Different approved delineation methods may be used to calculate the radius and area to be protected. The criteria used to delineate wellhead protection areas depend on the type of aquifer and the degree of protection desired. The criteria used include:

1. Distance from the well.

2. Drawdown of water table.

3. Time of travel.

Delineation of wellhead protection areas based on distance from well or a drawdown of the water table is most appropriate for shallow wells in unconfined aquifers [WWRDS, 2006 and Trautmann et al., 2005]. Protections of the area immediately surrounding the well will help prevent contamination from bacterial and viruses, do not provide complete protection from chemicals. Some of the chemicals and heavy metals can be transported for long distances underground without being degrade. Protection of the wells means the determination of the area to be protected. This area can be divided into zones to allow for varying degree of management relative to the sensitivity of each zone to groundwater contamination. Zones to be protect are divided into three sub-zones including: Accident Prevention Zone (Zone 1), Attenuation Zone (Zone 2), and; the Remedial Action Zone (Zone 3).

Zone 1, constitutes the accident prevention zone and is a highly protected area around the wellhead. Its purpose is to protect the annulus of the wells from the direct introduction of contaminants into the well and its immediate area from spills or leakage from storage facilities or containers.

Zone 2, the attenuation zone, is established to protect a well from contact with pathogenic mircoorganisms (bacteria, viruses) which can emanate from sources like landfill located close to the well [EPA, 1987].

Zone 3, is defined as the remedial action zone and is designed to protect the well from chemical contaminants that may migrate to the well. It includes a major portion of the recharge area. It should be large to provide adequate time to detect and respond to contaminate release.

The basic method in delineation is the radial distance from the well without considering the groundwater flow direction [EPA, 1998]. It includes the ***Arbitrary Fixed Radius (AFR)*** method as the baseline for the delineation of Zone 1. The boundary of Zone 1 is set at an arbitrary fixed radius of 100 or 300 ft radiuses from the well. This is the minimum radius recommend as a boundary of the Zone 1. Because of the simplicity of this method and unaccountability of aquifer properties and well characterization, weaknesses and uncertainties are high. Figure 5.28 shows a 500 ft (about 150 m) radius around each well.

Setback distance is another method in the delineation of the wellhead protection determination. Some of the criteria used in this method are:

- Well should be 50 ft from a storm sewer main.
- Well should be 600 ft radius from any gasoline or fuel oil storage.
- Well must be at least 1,000 ft from land application of municipal, commercial and industrial.
- Well must be 1,200 ft from any solid waste storage, transpiration and landfills.

Zone 2 and Zone 3 are delineated using Calculated Fixed Radius (CFR) on 2 (5 for Zone 3) year time of travel which would provide a minimum distance away from the well [Harman et al., 2001]. It is used to determine the circular areas through which groundwater will travel over some period of time. The travel time of groundwater from the edge of the circular to the well is dependent on the radius of the circles.

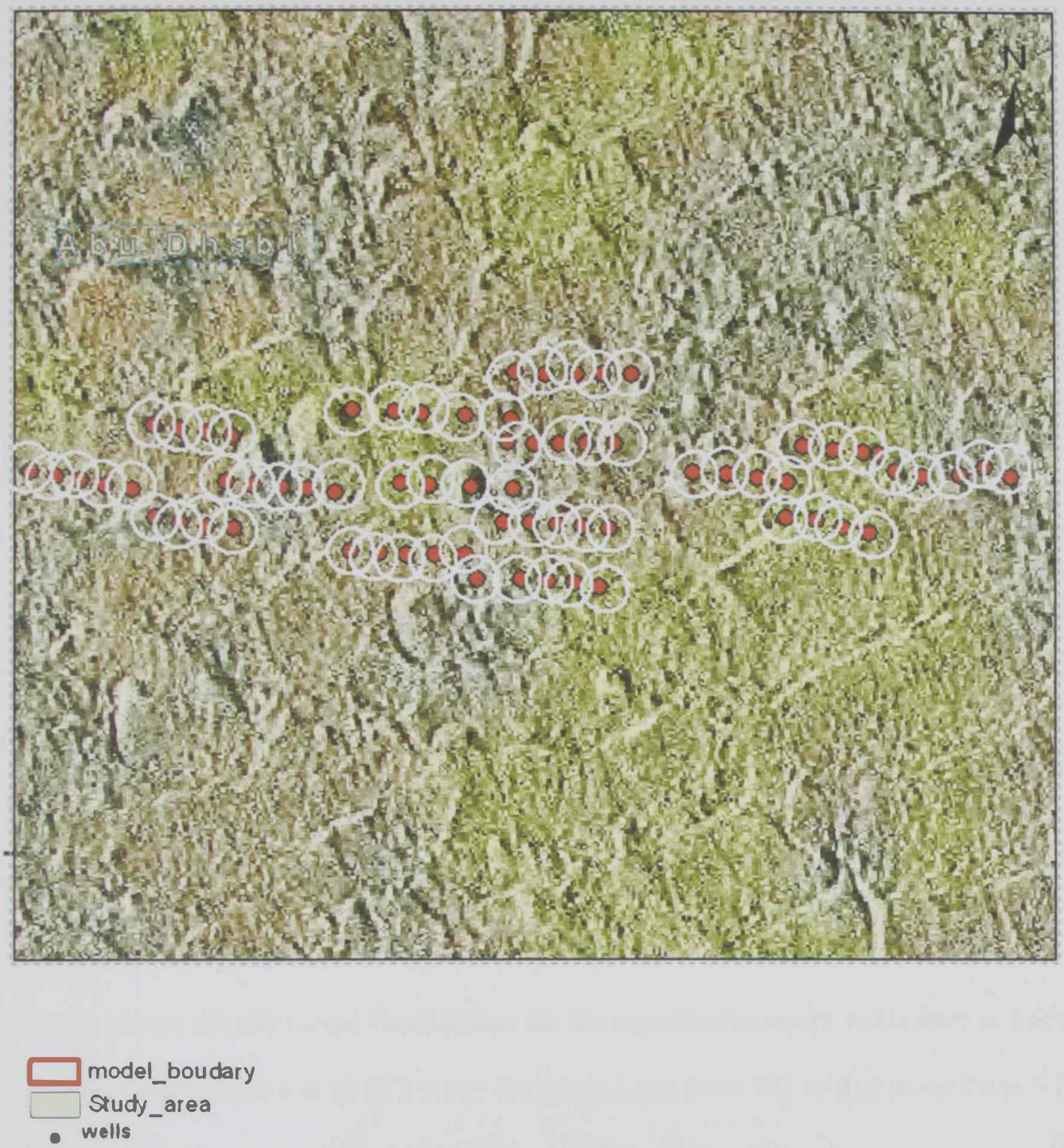

Figure 5.28 Arbitrary Fixed Radius around the wells in the study area

A specific formula is needed that utilizes pumping rate, pumping time, and recharge rate to calculate the specific wellhead protection area. This formula provides a radius (in feet) that indicates the distance outward from the centre of the wellhead which should be protected. It will also provide the area in square miles that needs protection. The formula used to calculate CFR is given as:

$$r_{min} = F.S\sqrt{\frac{Q\,t}{7.48\,n\,H\,\pi}}$$ [EPA, 1998 and WWRDS, 2006]

Where

r_{min} = radius of Zone 2 and Zone 3 measured from the well.

Q : Annual average pumping rate (gallons/yr)

t: Time of travel = 2 years for Zone 2 ; 5 years for Zone 3

n: average porosity

H : Screened or perforated interval of well (feet), actual length or 10 feet

F.S: Factor of safety; (to provide a buffer zone) and can be set equal to 1.3 (when all values are known) and 1.5 (when one or more values is not known).

Table 5.6 shows the calculated fixed radius for the injection/recovery wells used in this study. The radius ranges from 444 to 512 meter for Zone 2 and from 702 to 810 m for Zone 3. These criteria are considered in the study area (Figure 5.29).

The size and location of the groundwater protection zone could be defined by numerical modelling. Determinate the possible contamination sources and using the model to define the protection zone will determine how large these protection should be. The travel time approach is based on the estimated time of travel of groundwater or contaminate to the wells [Yacov and Haimes, 1984]. It is to draw a line in which groundwater is expected to reach the well within the 50, 100 and 365 days or more.

Table 5.6 Calculated fixed radius for delineation of Zone 2 and Zone 3.

No of wells	Duration [years]	Pumping rate [m^3/d]	Pumping rate gallon/yr	t= 2 years r min (m) ZONE 2	t= 5 years r min (m) ZONE 3
35 wells	1 st year	650	62,674,831	462	730
	2nd year	650	62,674,831	462	730
	3rd year	750	72,317,112	496	784
	4 th year	800	77,138,253	512	810
	5 th year	600	57,853,690	444	702
6 wells	2nd year	650	62,674,831	462	730
	3rd year	750	72,317,112	496	784
	4 th year	800	77,138,253	512	810
	5 th year	600	57,853,690	444	702
9 wells	3rd year	750	72,317,112	496	784
	4 th year	800	77,138,253	512	810
	5 th year	600	57,853,690	444	702
2 wells	4 th year	800	77,138,253	512	810
17 wells	5 th year	600	57,853,690	444	702

Since the rate of contaminate travel is likely to be equal to or lower that the rate of groundwater [Trautmann et al., 2005], this method is appropriate to estimate the contaminate travel time. The travel time of the groundwater required for the elimination of pathogenic bacteria is normally 50 days in Europe [UNEP, 2006]. Within this protection zone, no human being or even cattle is allowed to live. Therefore, a fence around this area is recommended. For the study area, this protection zone was determined through defining the area of freshwater around the wells that has a water level rise of 3, 4, 5 and 6 m after the end of the injection period. The distances corresponding to the 50, 100 and 365 days travel time within the groundwater were also included.

FEFLOW was used to determine the distance of the 50, 100 and 365 days time of travel. The particle tracking in FEFLOW computes the pathlines and isochrones of particle points within the computed flow field. There are two types of tracking; forward and backward as given hereafter.

+ **Forward:** A forward particle tracking is performed .i.e. the particles travel on the original flow direction.

+**Backward:** Particles in the flow fields travel in the inverted direction (backward) flow path direction. The number of element around the wells was refined to be less than 5 m in size to get more accurate results when using the particle tracking methods.

A refinement of the mesh around the wells was elaborated using Archmap. The result was used in FEFLOW as given in Figure 5.30. The area that is affected with 6, 5, 4, 3 m water level rise is shown in Figure 5.31. Determination of the protection zone will cover the 6 m water level rise and the protection of the wells for the 50, 100, 365 days (Figure 5.32). Background particle tracking is used to examine each potential source alone as shown in Figure 5.33 through 5.36, respectively. The possible distance of contamination to reach the well fields from all the sources is shown in Figure 5.37.

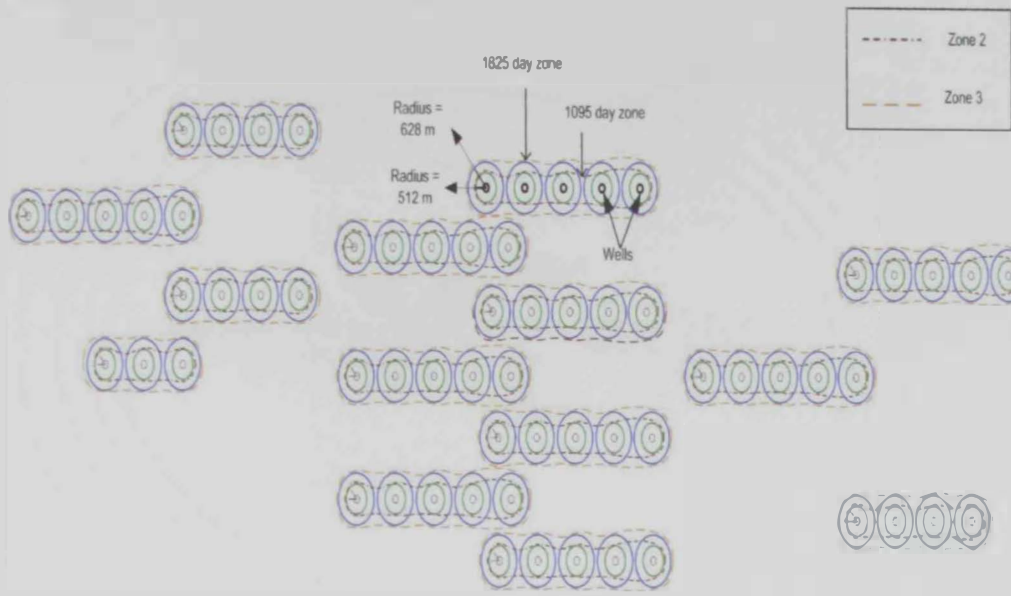

Figure 5.29 Delineation of Zone 2 and Zone 3 for the wells in the study area using CFR method

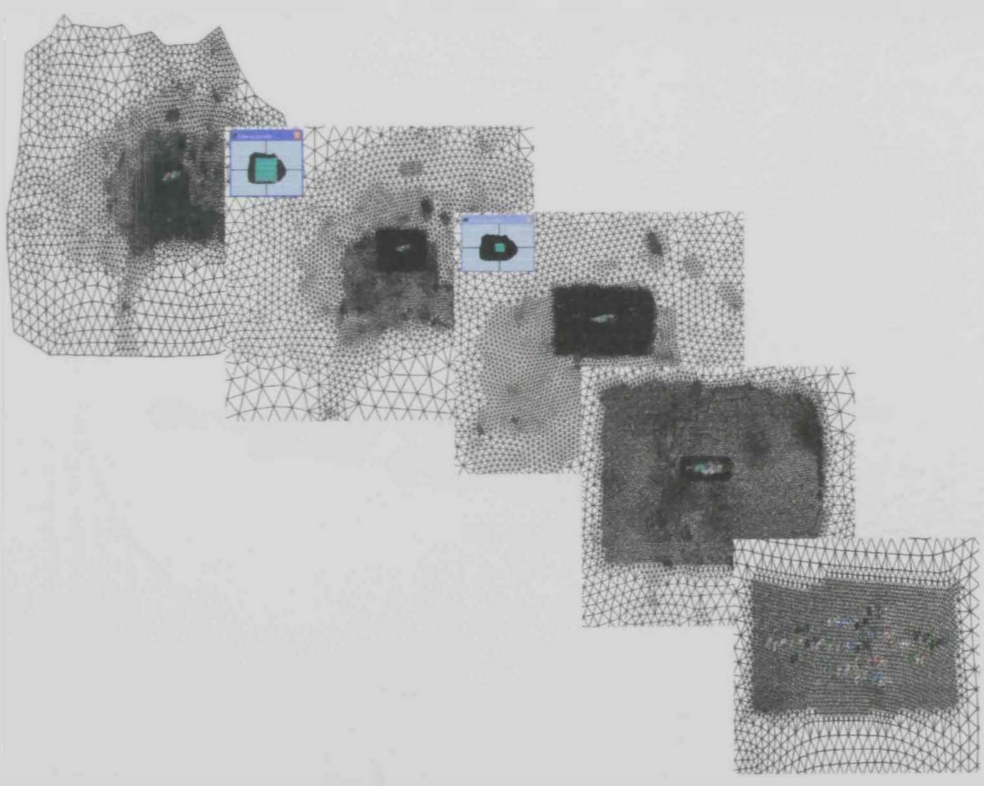

Figure 5.30 Refinement of the mesh for the contaminant study

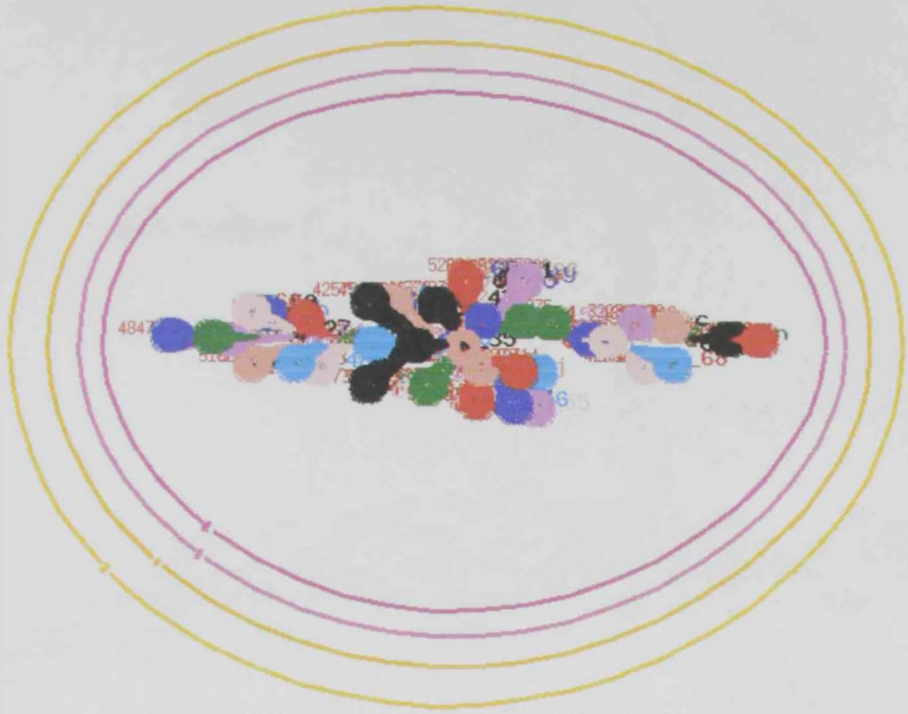

Figure 5.31 Accumulated difference of 3, 4, 5 and 6 m of groundwater level rise- at the end of recharge.

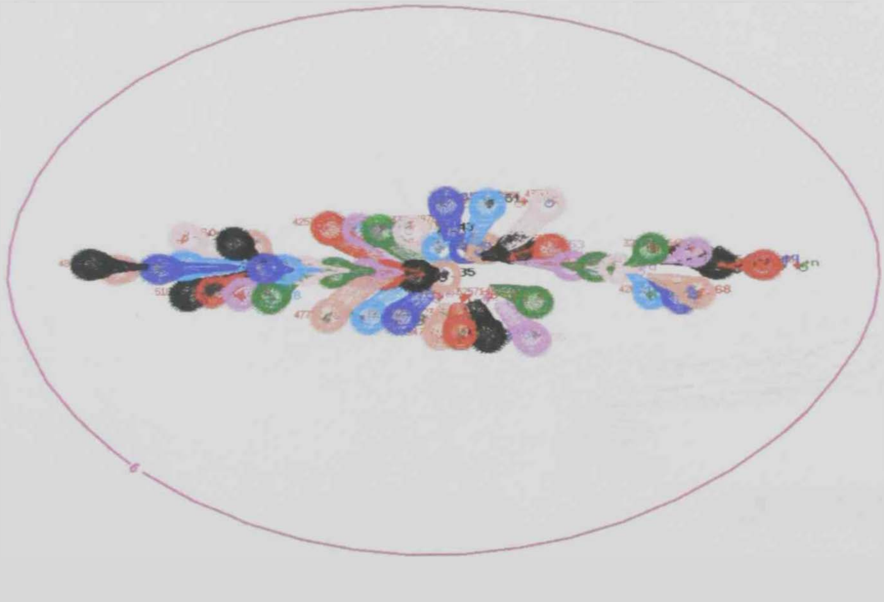

Figure 5.32 Accumulated 6 m of groundwater level rise in addition to the 50,100 and 365 days travel time.

Figure 5.33 Backward particle tracking for the 50 day travel of time - dumping area.

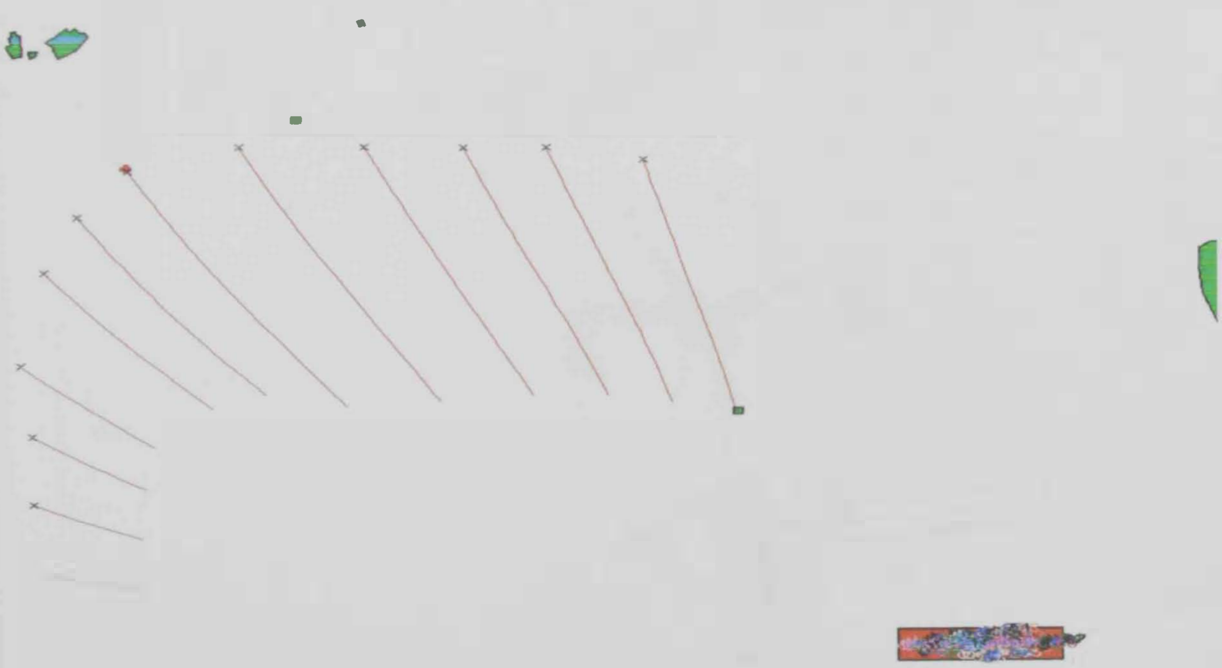

Figure 5.34 Backward particle tracking for the 50 day travel of time – Bu hasa Oil fields

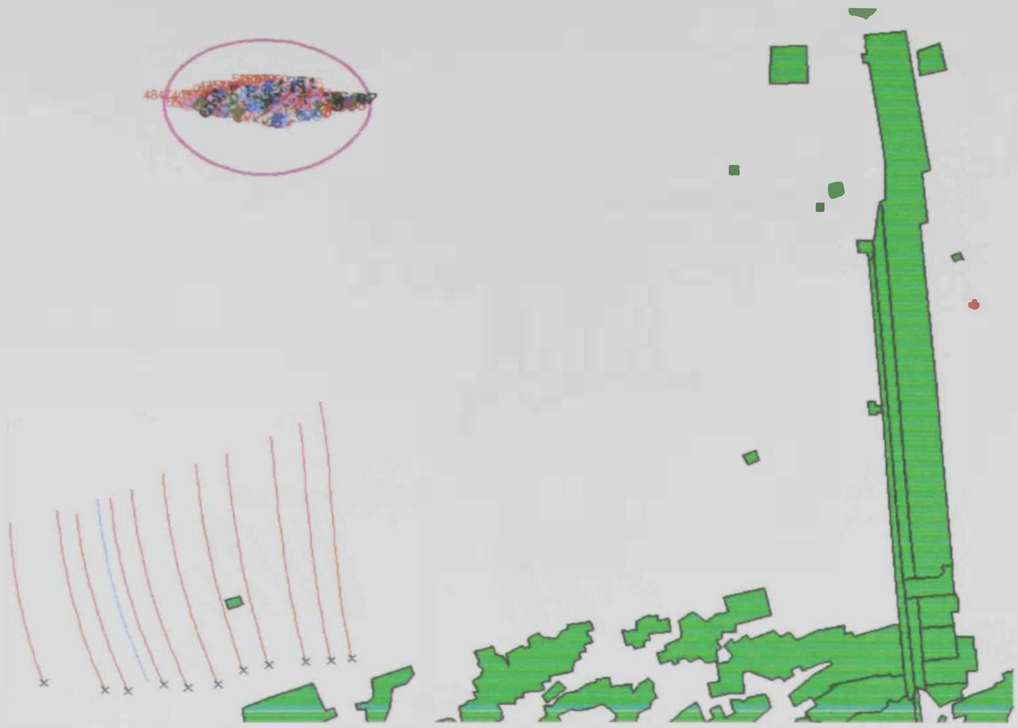

Figure 5.35 Backward particle tracking for the 50 and 100 days travel of time – Southern Petrol Station.

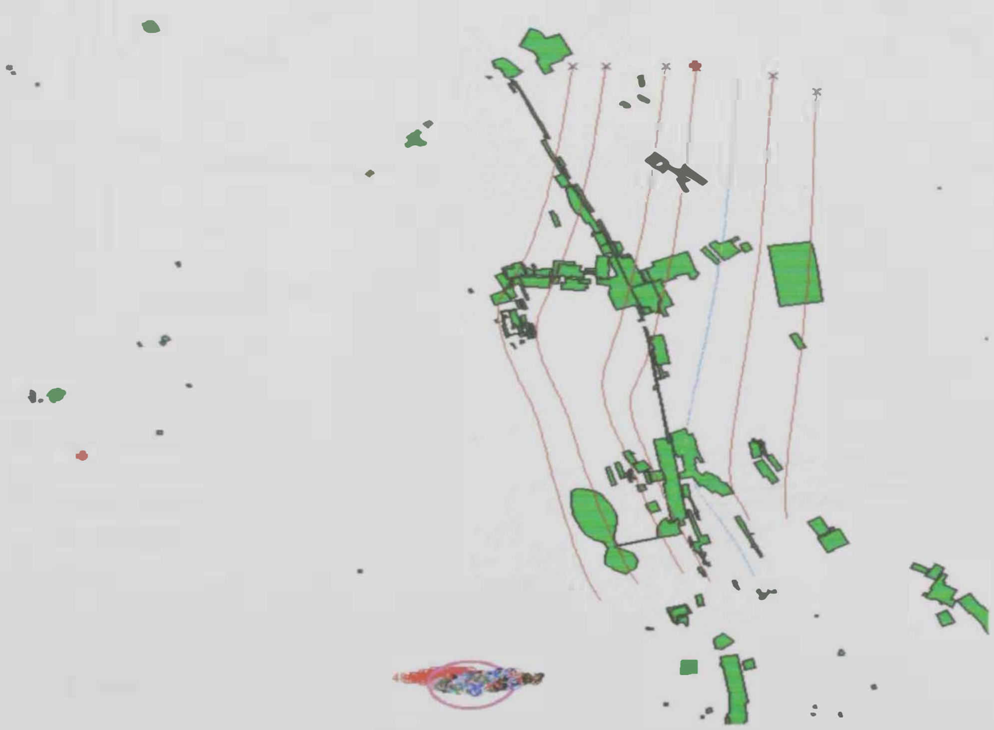

Figure 5.36 Backward particle tracking for the 50 and 100 days travel of time – Northern Petrol Station.

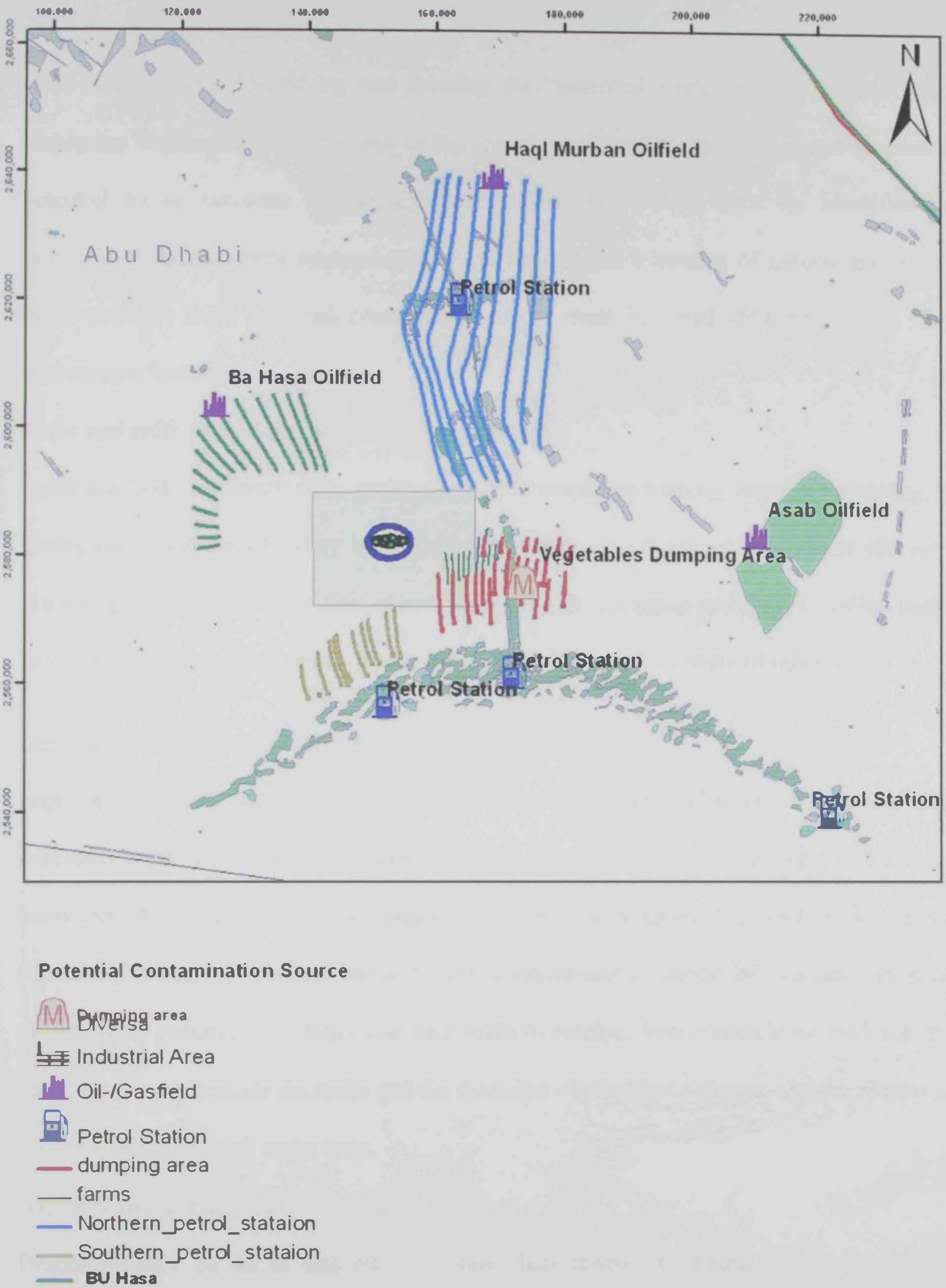

Figure 5.37 Particle tracking for all the potential contamination in the study area

Step 2: Contamination Inventory

This step includes identifying and locating the "potential contamination sources" found within the Wellhead Protection Area or the specific delineated area. Any source that has the potential to contaminate the groundwater within the WHPA must be identified and inventoried. Groundwater contaminations can occur from a number of natural and human-made sources. Each potential contaminant source must be listed [Masters, 1998]. These sources may include:

Leaks and spills at factories and commercial facilities

Spills and leaks can result from accidents, lack of employee training, improper planning, and inadequate maintenance. They are especially problematic if proper procedures are not in place to clean them up once they occur. Materials that can cause problems if spilled include gasoline, other petroleum products, hazardous chemicals, and a variety of other materials.

Improper hazardous waste disposal

Improper industrial waste disposal can come from a variety of sources, including major industrial plants and small businesses. The local dry cleaner uses a number of solvents and hazardous chemicals for cleaning clothes, and these must be handled as carefully as any other hazardous waste to prevent groundwater contamination. Industrial wastes can create groundwater pollution problems that take years to resolve. The contaminant evidence from the disposal may include the spills and the detection of chemical solvents, metals, nitrates and other chemicals in well water tests.

Improper use and disposal of pesticides

Pesticides used on farms and even on individual lawns can create serious groundwater pollution. Improper pesticide use can cause people and animals to become ill, kill plants, and have adverse effects on aquatic life in nearby streams. Improper pesticide use can include improper storage or improper disposal of excess pesticides.

Leachate from landfills

If landfills are not properly constructed, liquid from decomposition of materials, or leachate, can leak out of the landfill into an aquifer. Leachate can contain high levels of bacteria, hazardous chemicals, metals and ammonia. Runoff water from landfills after rains can also carry pollution to groundwater recharge areas and hence into groundwater. New landfill construction methods are designed to prevent pollution of groundwater. Landfills are now built with liners to prevent leachate from seeping through soil into aquifers. Leachate collection systems store the liquid away from the water table.

Fertilizers and pesticides

Overuse of fertilizers and pesticides is one source of groundwater pollution if it is being used without planning. Such overuse can allow nitrates from fertilizer to seep into the water table and pollute the groundwater. Careful use can avoid or minimize these problems especially if a high nitrate level in well water is observed during the water test.

Animal wastes

Animal wastes are sources of bacteria and nitrates. They can contaminate groundwater if too many animals are located in too small lot, or if the lot has improper drainage. Proper siting of animal lots, along with regular cleaning and avoiding overloading, can prevent animal waste pollution. Wastes can be recovered and used as fertilizer.

Leaking underground storage tanks

Leaking underground tanks are a potentially groundwater pollution source. Many of these tanks are old, corroded, and may leak and cause problems. Underground storage tanks are commonly found at service stations, where gasoline pollution is a potential problem.

Pipeline breaks

Pipeline breaks can be sources of localized groundwater pollution. Breaks can be severe enough so that they are immediately detected, or they may be small and cause significant groundwater contamination before they are noticed. Pipeline breaks can cause pollution from sewage, petroleum products, or other chemicals. They can occur around roadways due to vibration from vehicles, or they can even be caused by plant roots, which slowly crack pipes and cause leaks. Careful inspection of pipelines and regular maintenance can reduce pollution problems from this source.

Step 3: Implementation Stage Management

After the potential contaminant source inventory process, an implementation stage should take place. Two management approaches are generally used

Regulatory Management: Land use strategies, formulation of environment law, and subdivision regulations are all forms of regulatory management.

Non-Regulatory Management: This form of management may make use of numerous strategies including: public education, identifying the protection area, household hazardous waste collection, conservation easements, and land transfer (land purchase and/or land donation).

6 Conclusions and Recommendations

6.1 Conclusions

The main source of the fresh water in Abu Dhabi Emirate is the desalinated water. An appropriate back-up supply of water for crisis management is needed. The storage capacity of the desalination plants in the Emirate is enough for two days only of consumption. On the other hand, these plants are vulnerable to environmental hazards that may yield to a complete or partial shutdown.

Groundwater storage using the artificial recharge techniques is an alternative to provide the community with fresh water during possible crisis periods. The selected site ASR is located in Liwa area. The high quality of the native groundwater along with other geological consideration was the main reasons behind the selection of this site. A 90 days domestic water demand was considered in this study. A numerical model was applied to study the feasibility of the recharge and recovery in the study area. The results of the simulation show that the recharge and recovery of groundwater into existing fresh water aquifer is feasible and can be applied. About 70 % recovery was achieved during the simulation. The average natural groundwater level in the study area is 107 m. After five years of continuous recharge with the desalinated water with a total volume of 82.82 Mm^3, the groundwater level is expected to rise by 28 m. A total of 69 wells were used and were arranged in hexagonal pattern. The wells at the centre were highly affected by the recharge more than the ones at the edges. The groundwater level rise at the center reached 10 m. Neither the rise of water table nor the drawdown exceeded 4 m in the surrounding areas of the well fields.

Groundwater, under most conditions, is safe and more reliable for use than surface water. Groundwater is not as vulnerable as surface water, however, contaminates can still reach pumping wells. A potential pollution assessment for the study area was conducted and the potential contamination sources were identified. To determine whether the contamination sources may contaminate the shallow Liwa aquifer with specific references to the study area, different methods have been used to determine the well head protection zone. These methods include the Arbitrary Fixed Radius (AFR), Calculated Fixed Radius (CFR) and Time of Travel (TOT) using FEFLOW. Results from the above mentioned methods were analysed to assess the potential of contamination and to determine the protection zone. A radius equivalent to 2 year travel time for the well fields ranged from 444 to 512 m while for Zone 3 the radius ranged between 700 and 810 m.

Backward particle tracking was used in FEFLOW. The calculated travel distance after the 50 and 100 days of injection from the potential sources to well fields shows that the study area is safe and would not be affected from any pollution sources.

6.2 Recommendations

1. Alternative water resources in Abu Dhabi Emirate should be considered to meet the demands under emergency conditions.
2. A feasibility study of recharging the aquifer with treated wastewater is recommended, although some constraints should be taken into account where the groundwater in the emirate is used for the domestic uses .
3. A complete network of water transmission between all ADWEA desalination plants should be considered. The excess of water from any plants should be directly used for the recharge of the aquifer.
4. Educational awareness program of public toward the conservation of water resources should be implemented.

5. Regulation management of the water supply and artificial recharge site should be considered to determine the land use near the site and ensure proper use of the fertilizers in farms near the site.

6. Remoteness of the recharge site should be taken into consideration for the future projects.

7. A monitoring system for groundwater levels and quality should be extended to the whole Emirate.

8. Compressive studies regarding water demands and production from desalination plants are needed. Database for all of the information could be established.

9. A management plan to reduce potential contamination from possible pollution sources and a contingency plan for emergencies are needed

10. Owners and operators of facilities that use or store hazardous materials should be notified that they are in a wellhead protection areas if applicable. They should be encouraged to use best management practices to prevent groundwater contamination.

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Appendices

Appendix- I

Aquifer Thickness

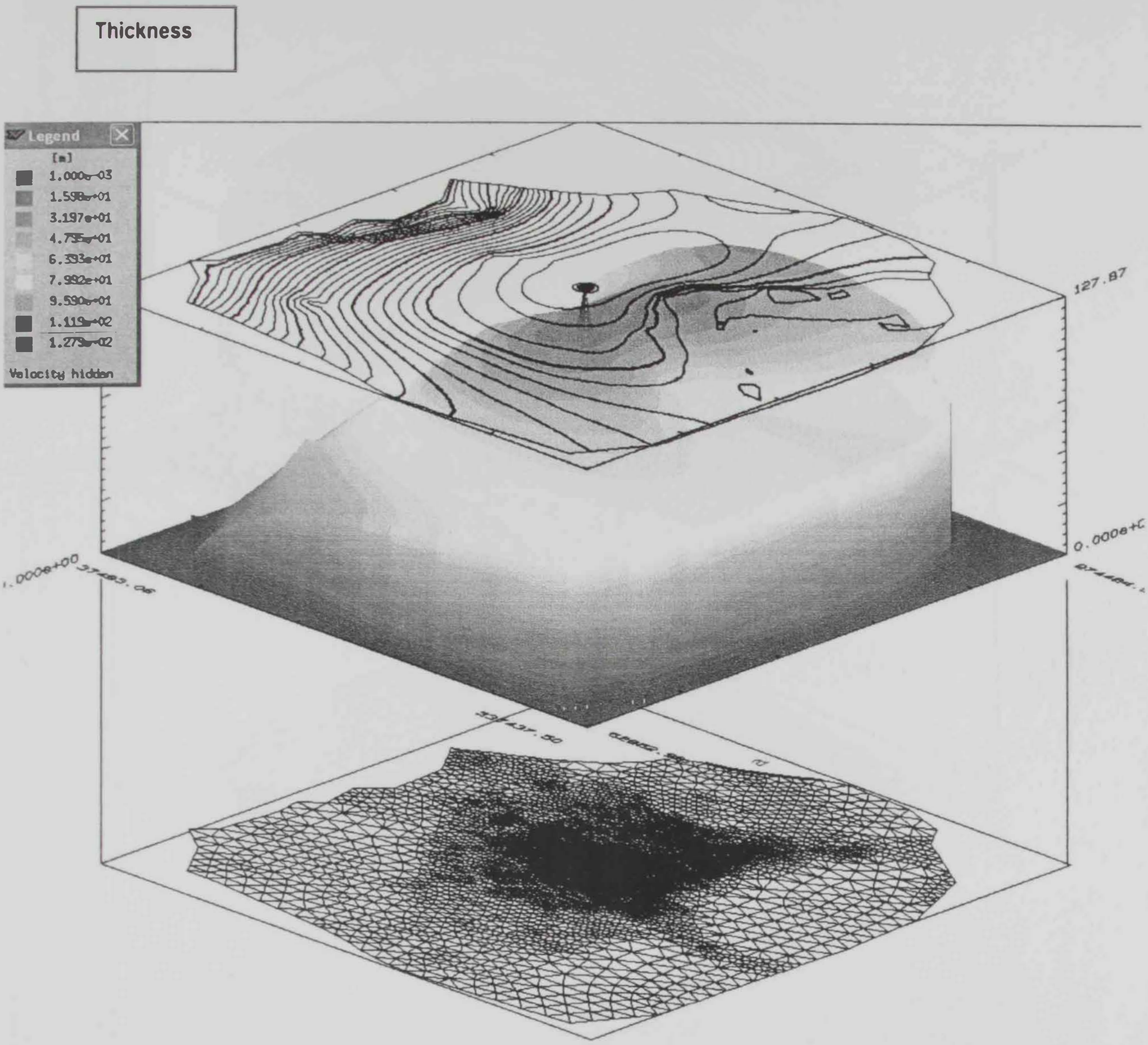

Figure 1. 3 D projection of the thickness (Slice _1)

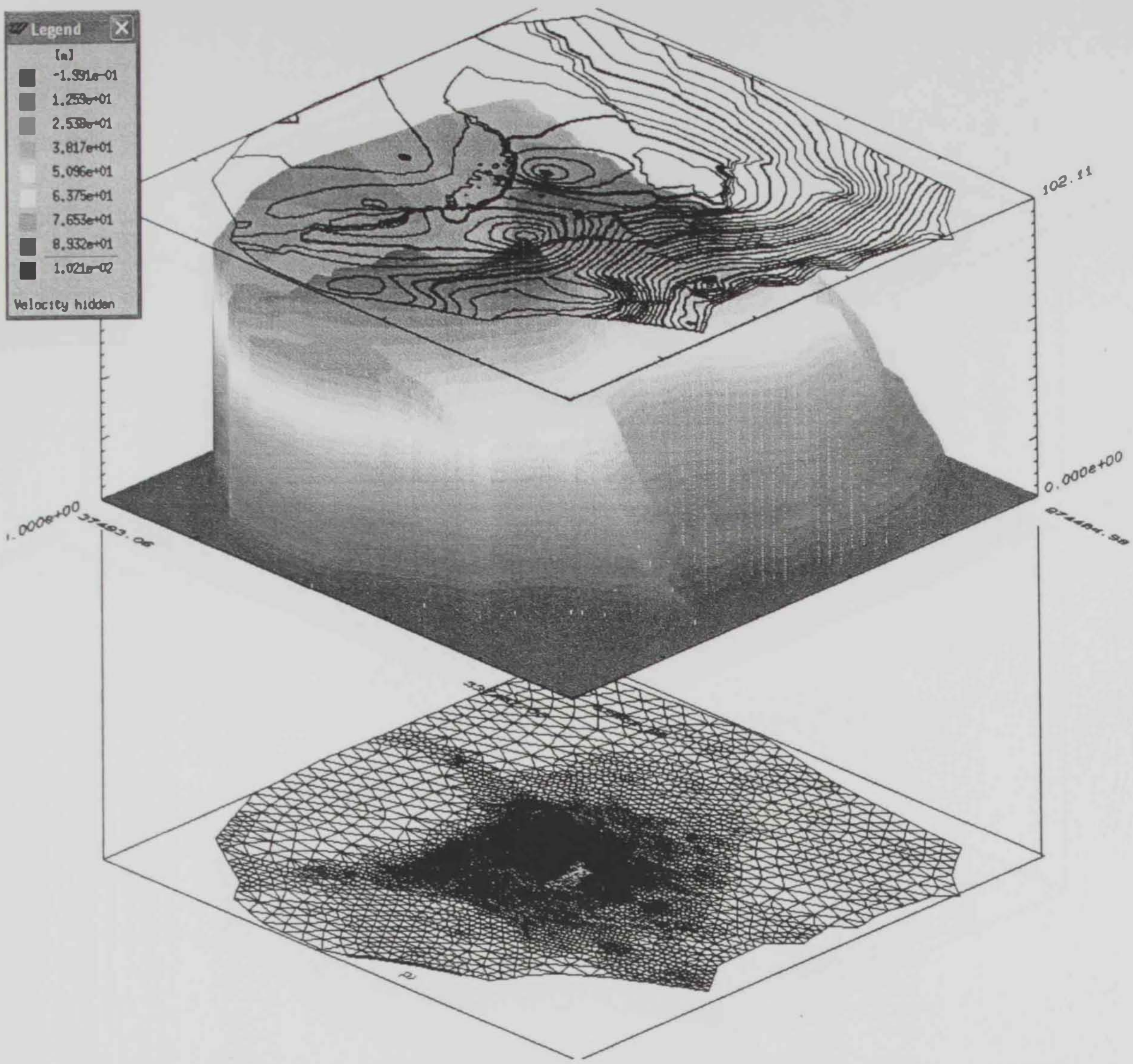

Figure 2. 3 D projection of the thickness (Slice _2)

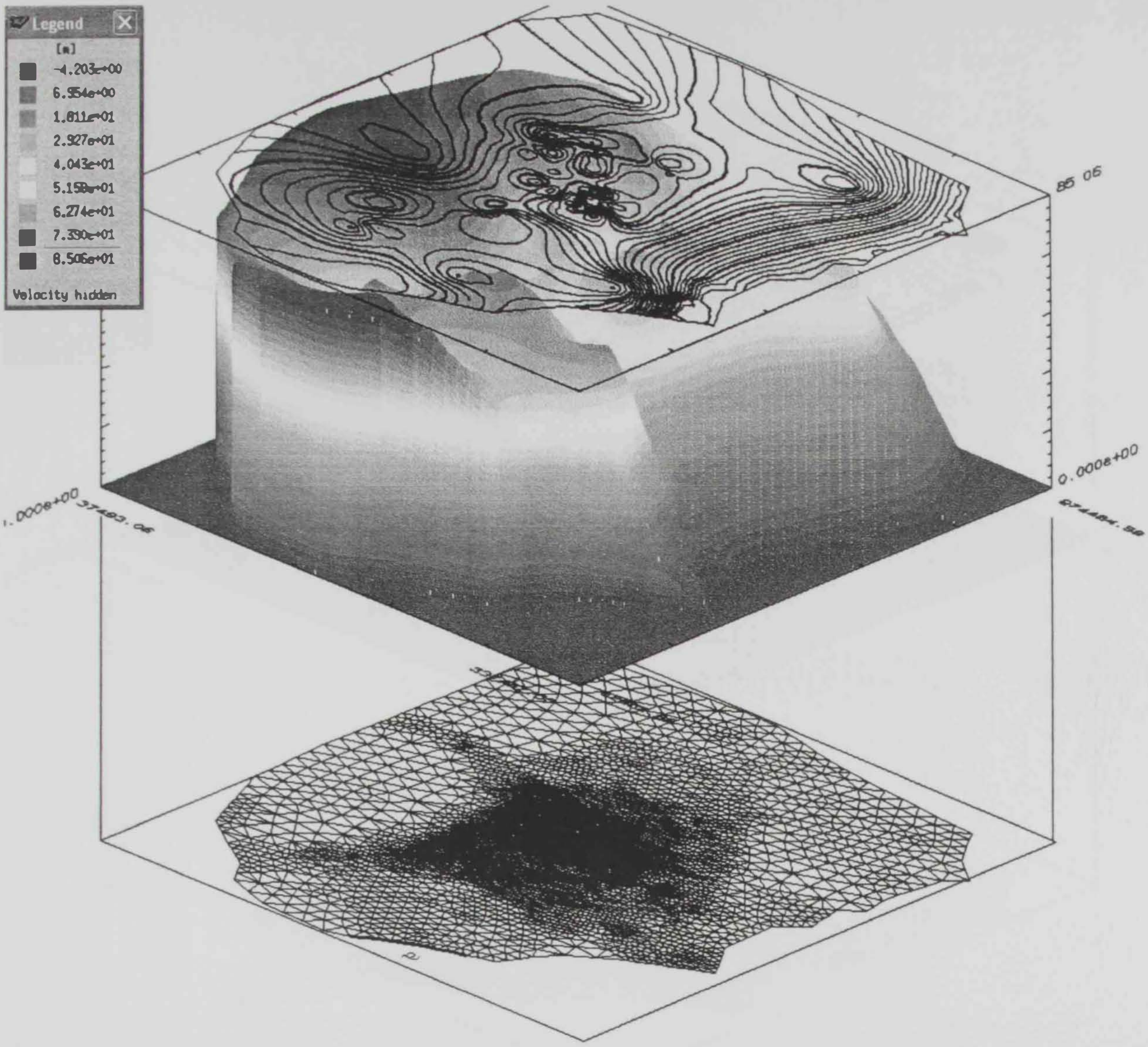

Figure 3. 3 D projection of the thickness (Slice _3)

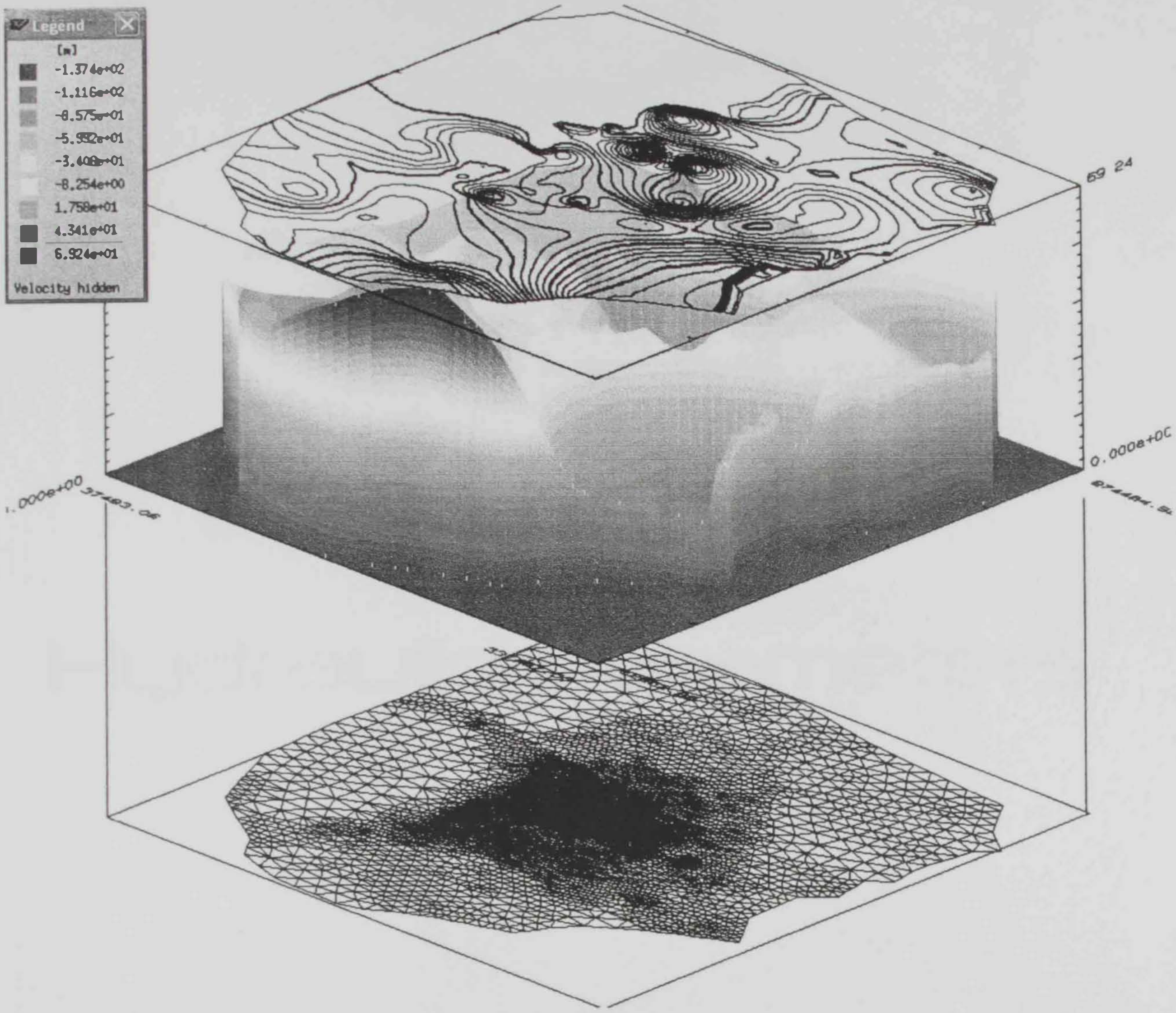

Figure 4. 3 D projection of the thickness (Slice _4)

Appendix- II

Hydraulic parameters

Hydraulic conductivity of the principle x –direction

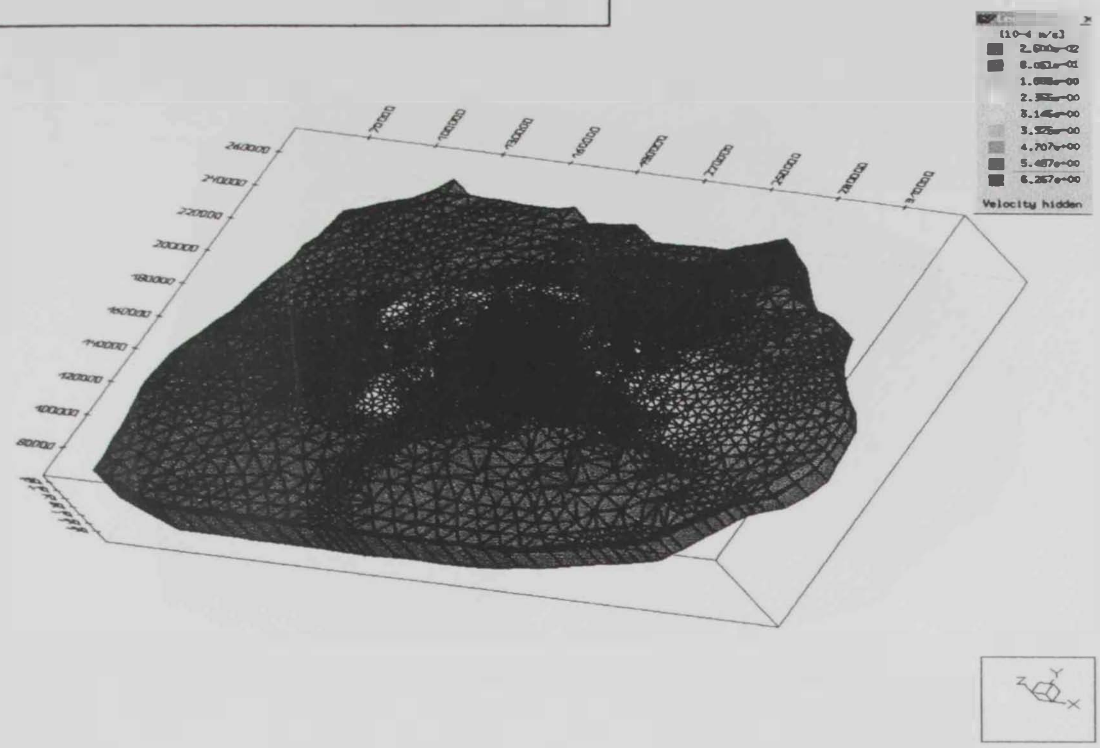

Figure.1 3 D of the Hydraulic conductivity of the principle x –direction

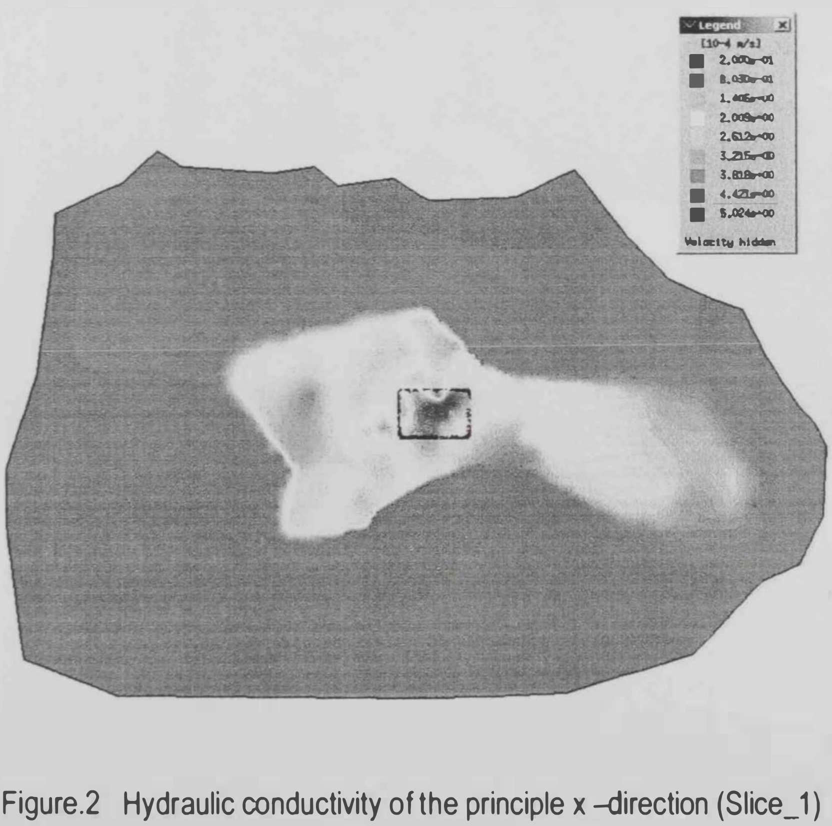

Figure.2 Hydraulic conductivity of the principle x –direction (Slice_1)

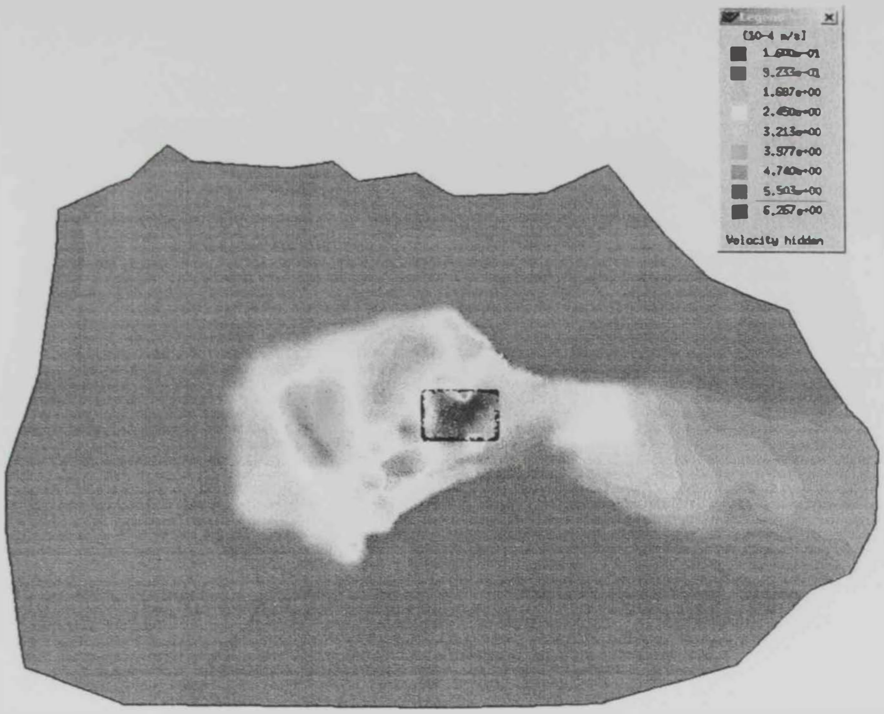

Figure 3 Hydraulic conductivity of the principle x –direction (Slice_2)

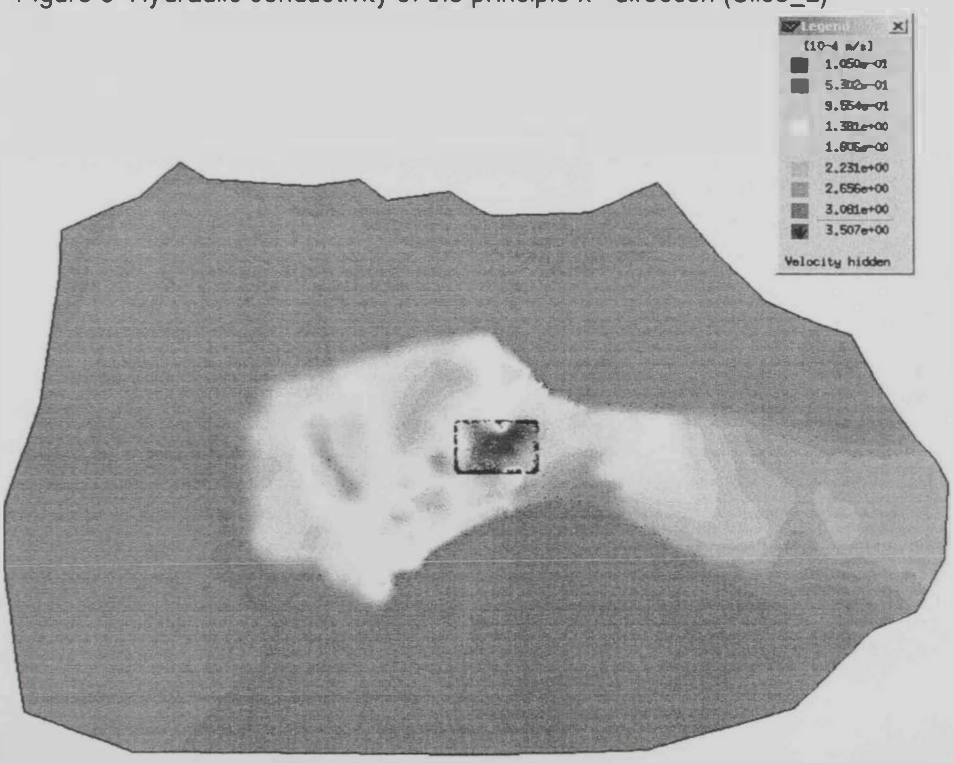

Figure 4 Hydraulic conductivity of the principle x –direction (Slice_3)

Figure 5 Hydraulic conductivity of the principle x –direction (Slice_4)

Hydraulic conductivity of the principle y –direction

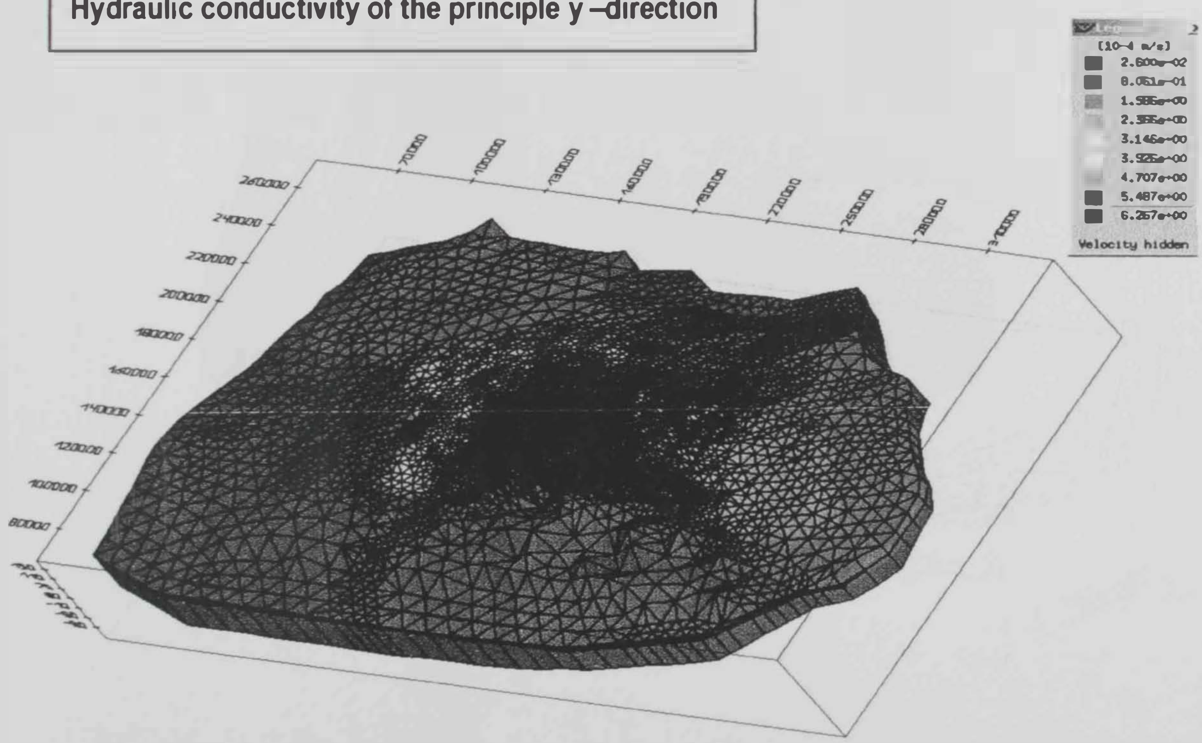

Figure 6 3 D view of the hydraulic conductivity

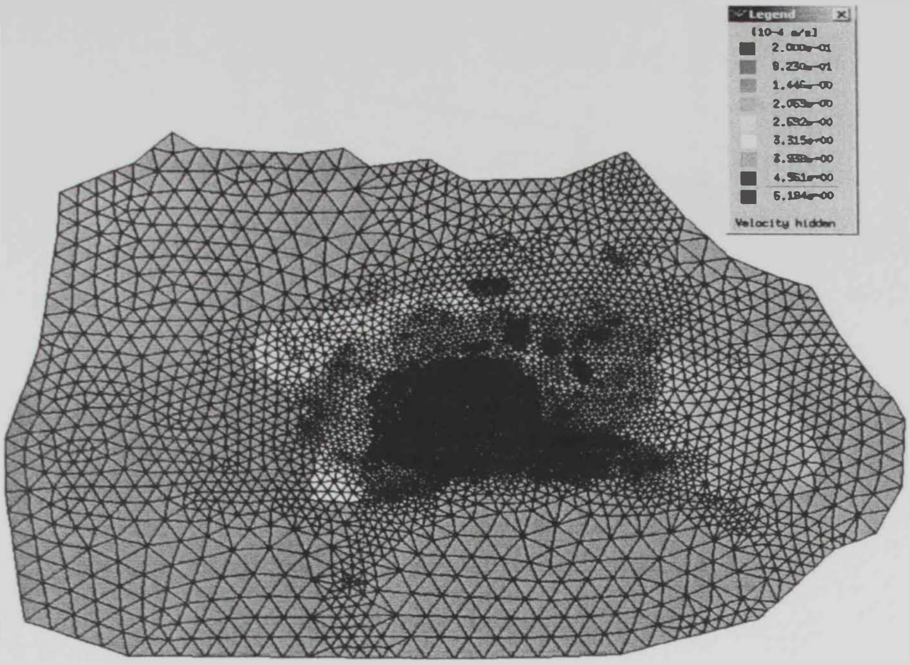

Figure 7 Hydraulic conductivity of the principle y –direction (Slice_1)

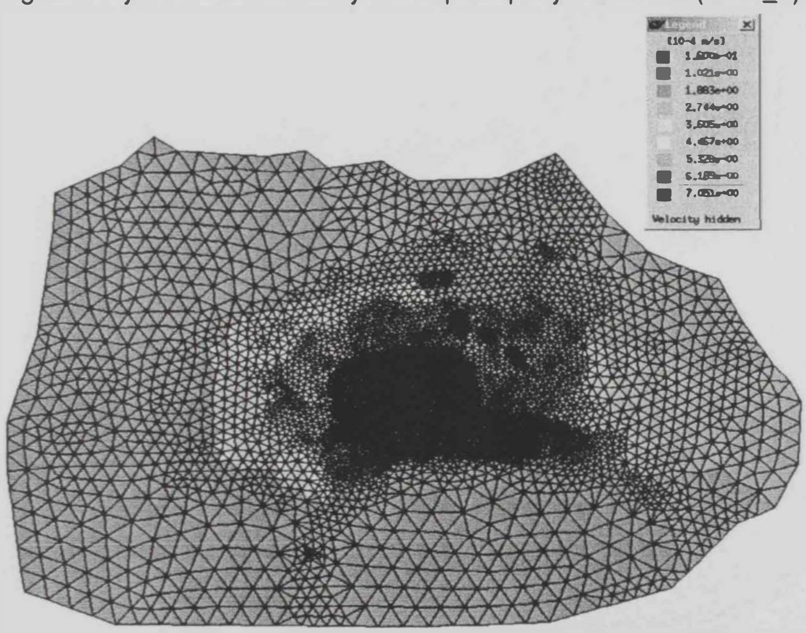

Figure 8 Hydraulic conductivity of the principle y –direction (Slice_2)

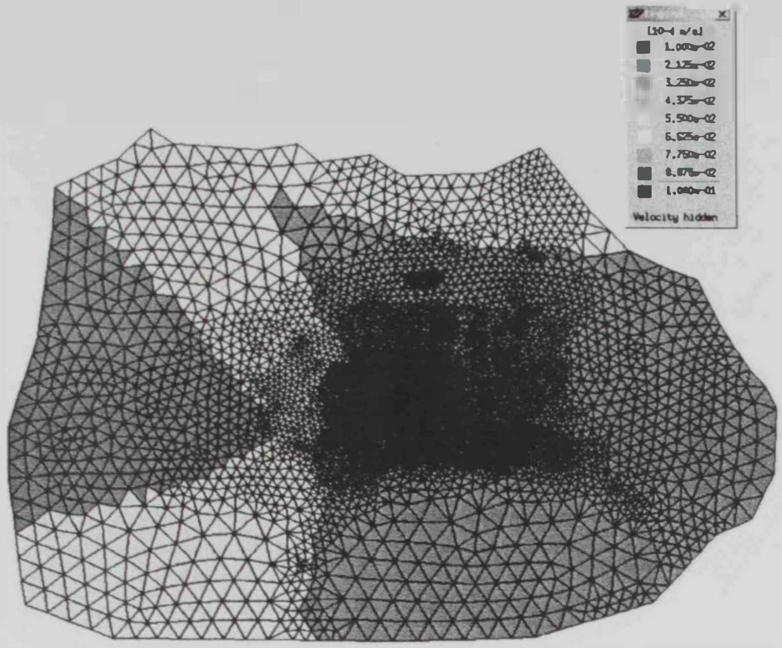

Figure 9 Hydraulic conductivity of the principle y –direction (Slice_3)

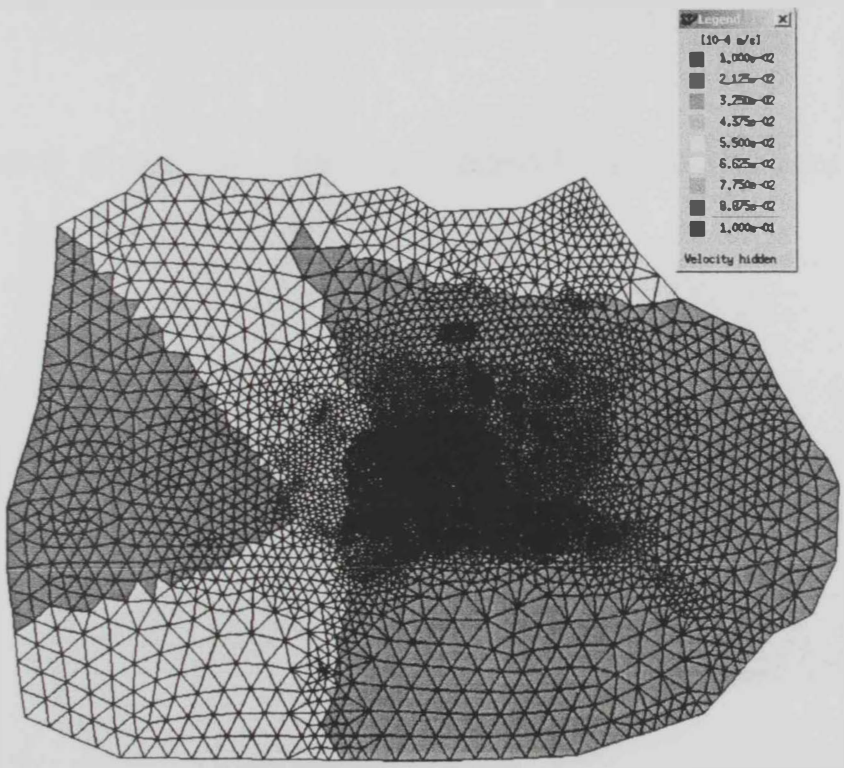

Figure 10 Hydraulic conductivity of the principle y –direction (Slice_4)

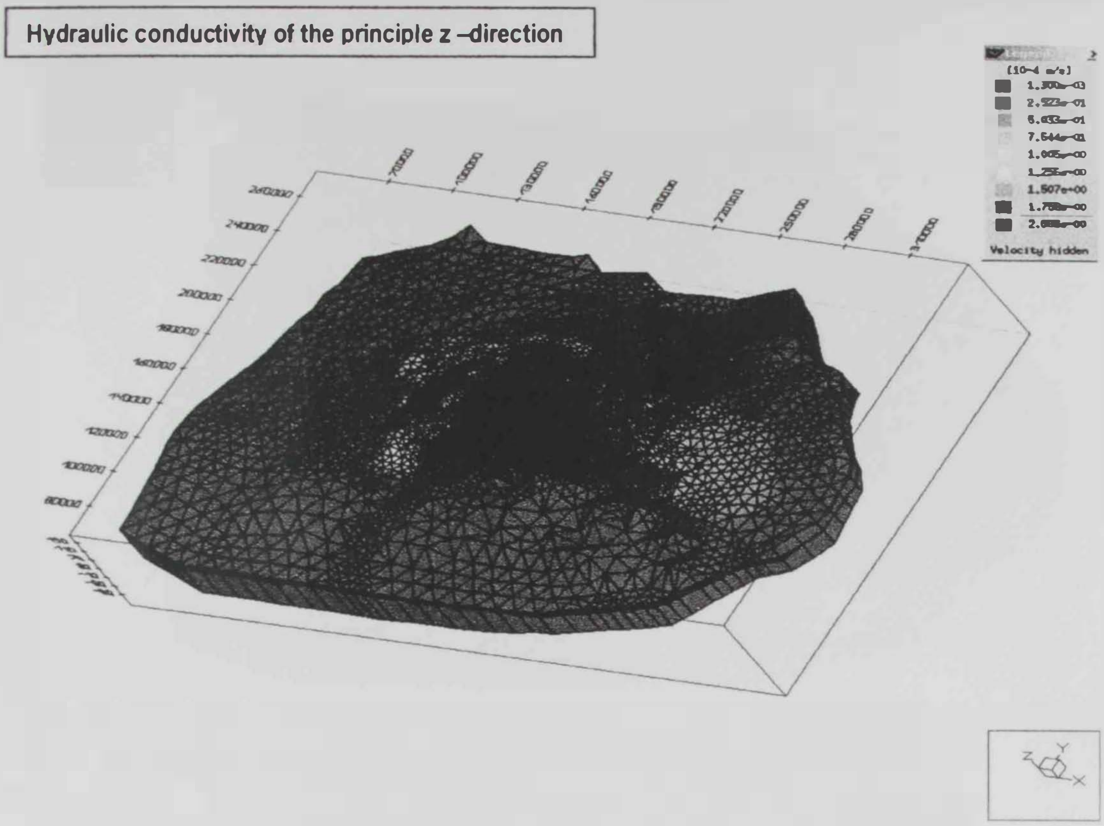

Figure 11 3D view of the hydraulic conductivity of the principle z –direction

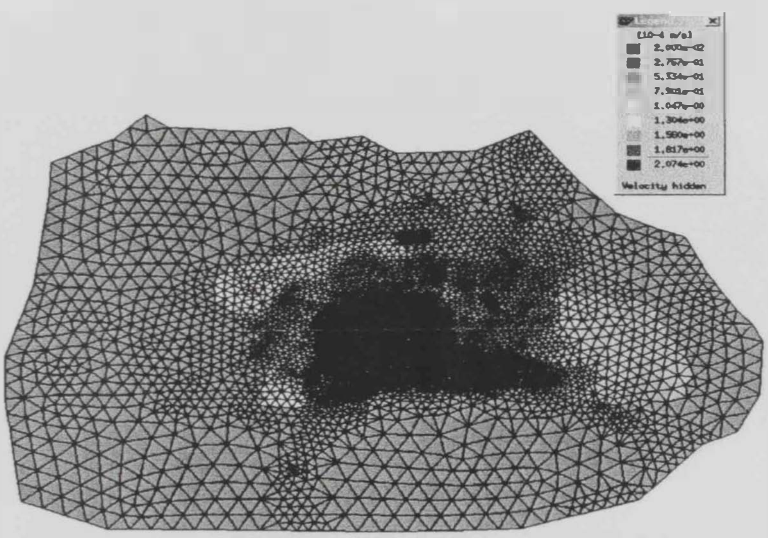

Figure 12 Hydraulic conductivity of the principle z –direction (slice_1)

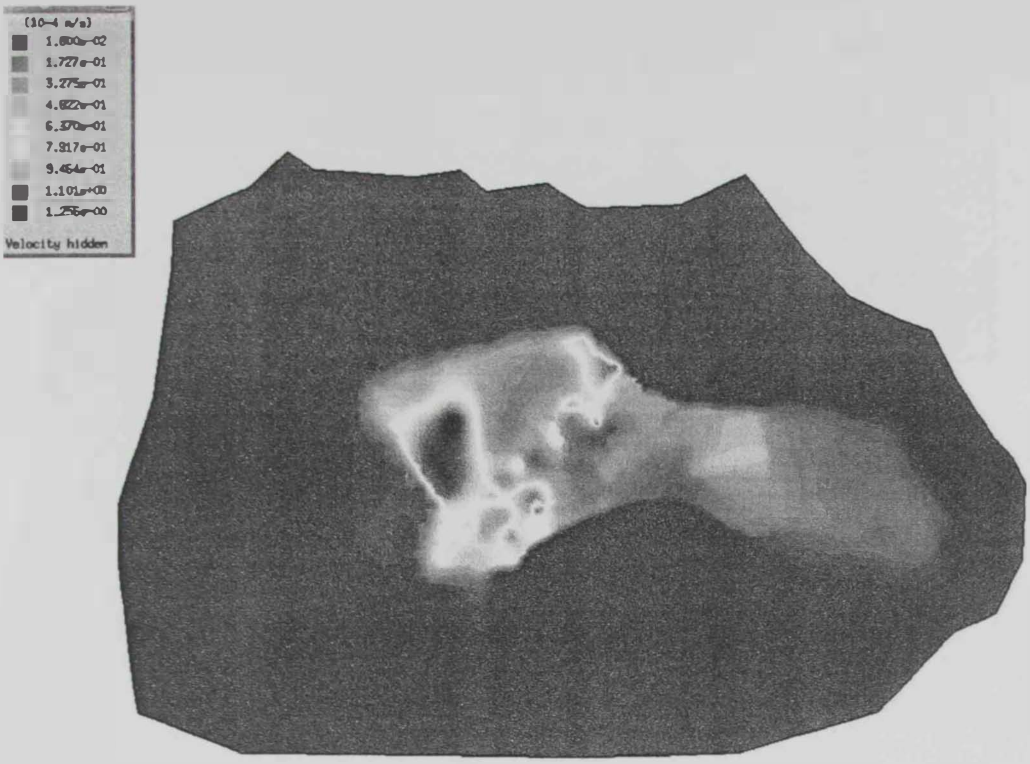

Figure 13 Hydraulic conductivity of the principle z –direction (slice_2)

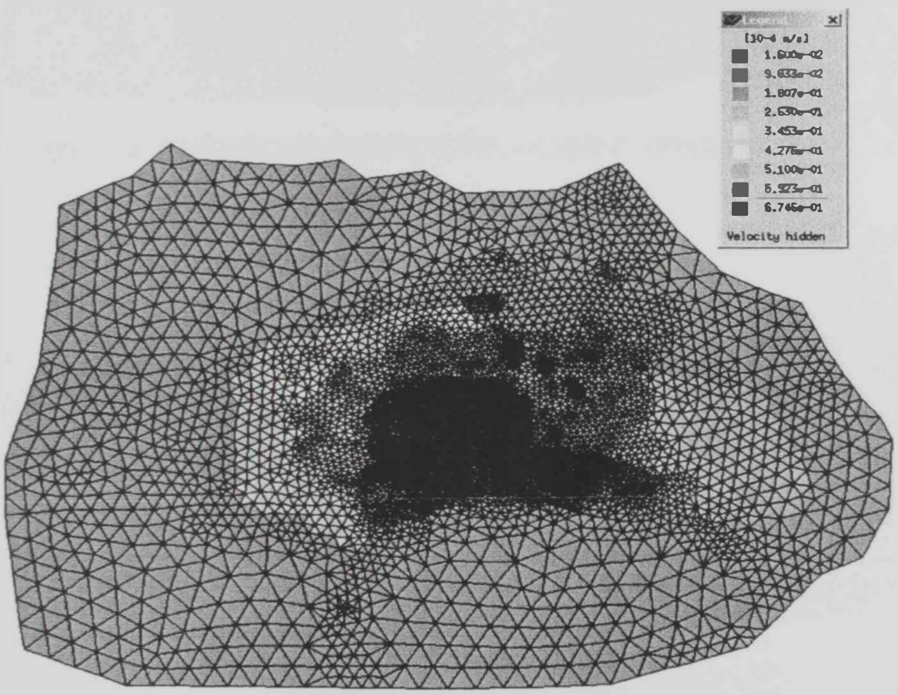

Figure 14 Hydraulic conductivity of the principle z –direction (slice_3)

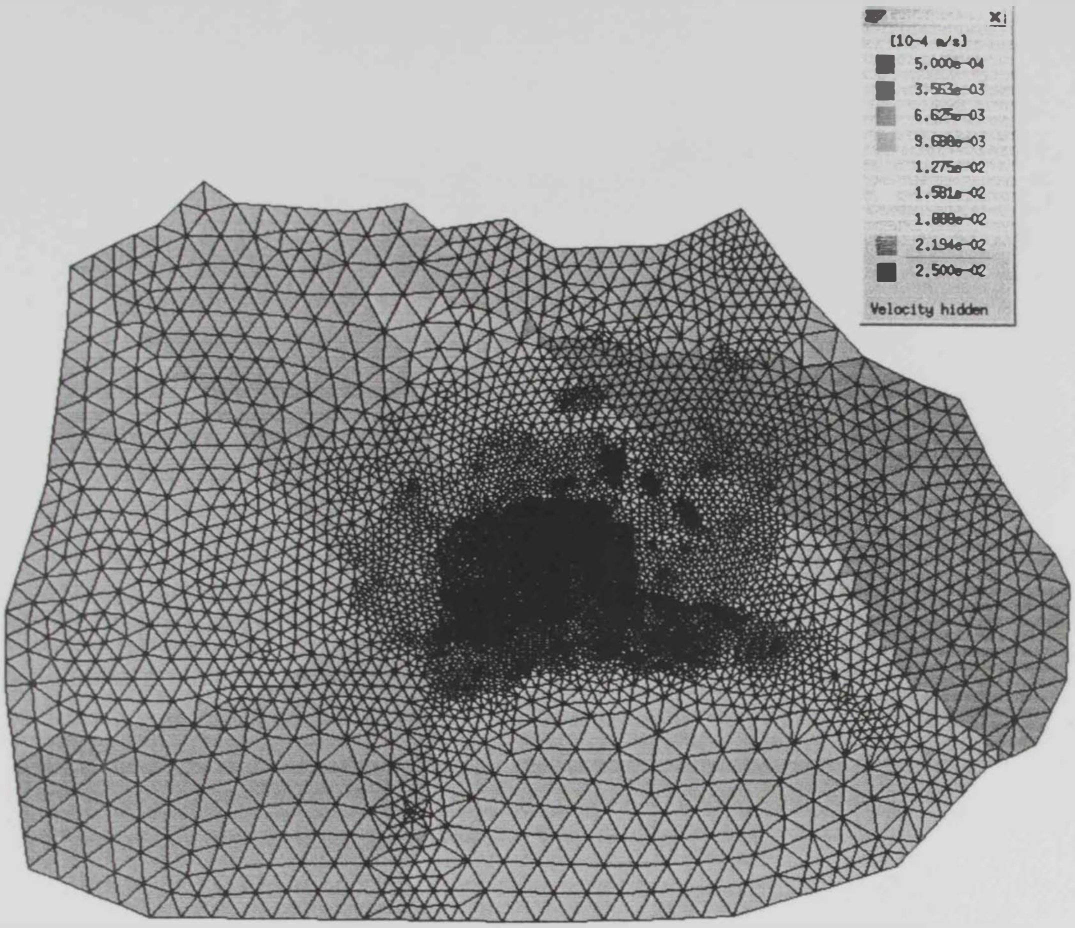

Figure 15 Hydraulic conductivity of the principle z –direction (slice_4)

Appendix- III

Results of calibration of the model

Table.1 Wells used in the calibration process and differences between simulated and measured heads [m]

Well ID	X40	Y40	Measured Head [m]	Date	Simulated Head [m]	Difference [m]
ARL-OB-19	167,323	2,584,596	103.92	12/05/03	103.13	0.87
ARL-OF-17	161,661	2,581,691	107.14	12/05/03	103.13	3.87
GOW-103	75,901	2,573,917	87.50	12/05/03	90.24	-2.24
GOW-40-SUPPLY	145,351	2,539,022	86.88	11/30/03	83.79	3.21
GOW-41	172,203	2,568,372	95.80	12/05/03	99.90	-3.90
GOW-42	224,949	2,555,642	97.58	12/05/03	93.46	4.54
GOW-43	249,218	2,592,683	103.72	12/05/03	99.90	4.10
GOW-43-SUPPLY	249,276	2,592,707	103.70	12/10/03	99.90	4.10
GOW-44	171,198	2,589,596	104.01	12/05/03	103.13	0.87
GOW-45	95,325	2,548,925	92.37	12/05/03	93.46	-1.46
GOW-46	187,370	2,559,005	88.64	12/05/03	87.01	1.99
GOW-47	132,001	2,545,319	89.85	11/29/03	90.24	-0.24
GOW-48	217,949	2,539,011	85.36	12/05/03	83.79	1.21
GOW-51	100,801	2,535,093	91.32	12/05/03	90.24	0.76
GOW-54	73,624	2,539,382	87.63	12/05/03	87.01	0.99
GOW-58	128,620	2,531,633	88.46	11/29/03	87.01	0.99
GOW-61	201,366	2,521,350	78.09	12/05/03	74.12	3.88
GOW-62	232,471	2,519,163	75.71	12/05/03	70.90	5.10
GOW-81	152,298	2,552,784	91.47	11/29/03	90.24	0.76
GOW-84	193,838	2,588,912	105.49	12/06/03	103.13	1.87
GOW-92	137,090	2,603,998	98.91	12/05/03	96.68	2.32
GOW-95	81,182	2,547,852	89.43	11/30/03	90.24	-1.24
GOW-96	151,835	2,557,723	89.50	11/29/03	93.46	-3.46
GOW-98	83,944	2,541,392	89.69	11/30/03	90.24	-0.24
GWA-141	159,208	2,581,296	107.09	12/05/03	103.13	3.87
GWA-144	157,787	2,580,899	107.03	12/05/03	103.13	3.87
GWA-148B	151,358	2,577,915	106.53	12/05/03	103.13	3.87
GWA-151	163,739	2,582,885	106.83	12/05/03	103.13	3.87
GWA-153	146,539	2,579,906	106.35	11/30/03	103.13	2.87
GWA-156	156,004	2,576,076	165.00	11/30/03	103.13	2.87
GWA-158	157,233	2,572,846	105.66	12/05/03	103.13	2.87
GWA-162	156,517	2,571,241	104.96	11/30/03	99.90	5.10
GWA-166	152,862	2,568,267	103.48	12/05/03	99.90	3.10
GWA-167	152,277	2,572,864	105.25	11/30/03	103.13	1.87
GWA-172	159,461	2,585,580	106.64	12/01/03	103.13	3.87
GWA-177	137,745	2,579,476	105.55	11/30/03	103.13	2.87
GWA-178	142,612	2,577,105	88.92	12/05/03	103.13	2.87
GWA-18	102,699	2,585,828	105.00	12/16/03	93.46	4.54
GWA-181	134,285	2,575,988	104.73	12/05/03	103.13	1.87
GWA-184	145,642	2,599,331	102.02	12/05/03	99.90	2.10
GWA-187	132,553	2,580,156	104.94	11/30/03	103.13	1.87

GWA-189	152,483	2,562,623	94.18	11/30/03	96.68	-2.68
GWA-19	106,564	2,582,671	98.97	12/05/03	96.68	2.32
GWA-190	146,331	2,594,238	104.12	12/05/03	103.13	0.87
GWA-192	122,880	2,580,832	103.15	12/17/03	99.90	3.10
GWA-199 SUPPLY	126,378	2,589,322	103.25	11/29/03	99.90	3.10
GWA-200	112,467	2,581,719	100.45	11/29/03	96.68	3.32
GWA-201	129,253	2,573,844	103.70	11/29/03	99.90	4.10
GWA-204B	262,718	2,550,481	93.65	12/08/13	90.24	3.76
GWA-205	96,101	2,579,467	95.37	11/29/03	93.46	1.54
GWA-206	105,239	2,562,838	96.75	11/29/03	96.68	0.32
GWA-207B	127,883	2,580,284	104.25	11/30/03	103.13	0.87
GWA-209	120,078	2,563,033	99.65	11/30/03	96.68	3.32
GWA-210-SUPPLY	182,383	2,587,078	104.31	12/05/03	103.13	0.87
GWA-211-SUPPLY	112,545	2,571,466	99.74	11/29/03	96.68	3.32
GWA-212	254,133	2,545,882	92.99	12/05/03	90.24	2.76
GWA-213	109,335	2,604,518	95.06	12/05/03	93.46	1.54
GWA-214	124,261	2,567,866	101.99	11/29/03	99.90	2.10
GWA-215B	142,195	2,582,059	106.13	12/05/03	103.13	2.87
GWA-216	108,528	2,588,837	98.85	11/29/03	96.68	2.32
GWA-218	100,236	2,573,808	96.14	12/05/03	93.46	2.54
GWA-219	115,331	2,561,066	98.30	11/30/03	96.68	1.32
GWA-220	248,809	2,546,137	92.53	12/08/03	90.24	2.76
GWA-222	103,640	2,602,903	94.49	11/29/03	93.46	0.54
GWA-223	105,303	2,553,817	94.97	11/30/03	93.46	1.54
GWA-224	130,335	2,551,340	96.10	11/30/03	93.46	2.54
GWA-225	242,958	2,587,943	103.27	12/05/03	99.90	3.10
GWA-226	85,666	2,596,902	90.27	11/29/03	90.24	-0.24
GWA-229	110,345	2,559,145	96.96	11/30/03	96.68	0.32
GWA-231	135,819	2,559,275	100.31	12/05/03	96.68	3.32
GWA-232	177,304	2,565,804	101.57	12/05/03	96.68	5.32
GWA-233	94,508	2,603,843	91.98	11/29/03	90.24	1.76
GWA-235	240,792	2,548,707	93.82	12/08/03	90.24	3.76
GWA-236	140,161	2,561,165	101.01	11/30/03	96.68	4.32
GWA-237	96,267	2,598,590	93.84	10/14/03	90.24	3.76
GWA-239	118,834	2,540,535	93.07	12/05/03	93.46	-0.46
GWA-242	125,344	2,549,705	96.04	11/30/03	93.46	2.54
GWA-248	111,576	2,539,334	92.75	11/30/03	93.46	-0.46
GWA-249	184,531	2,576,426	107.16	12/05/03	103.13	3.87
GWA-251	116,762	2,545,199	94.34	11/30/03	93.46	0.54
GWA-253	178,878	2,582,221	106.43	12/05/03	103.13	2.87
GWA-254	146,490	2,589,810	106.81	12/05/03	103.13	3.87
GWA-256	151,532	2,596,869	103.31	12/05/03	103.13	-0.13
GWA-259	192,745	2,570,094	105.41	12/05/03	99.90	5.10
GWA-261	155,409	2,601,213	101.61	12/05/03	99.90	2.10

GWA-263	198,095	2,566,655	103.99	12/06/03	99.90	4.10
GWA-264	150,998	2,605,006	99.63	12/05/03	99.90	0.10
GWA-265	146,072	2,604,695	99.64	12/05/03	99.90	0.10
GWA-269	124,910	2,560,250	99.70	11/29/03	96.68	3.32
GWA-271	146,119	2,609,776	96.76	11/29/03	96.68	0.32
GWA-272	193,823	2,564,668	102.79	12/05/03	96.68	6.32
GWA-273	117,543	2,550,223	95.77	11/30/03	93.46	2.54
GWA-274	141,813	2,606,931	98.36	12/05/03	96.68	1.32
GWA-276	122,968	2,555,157	97.90	11/30/03	96.68	1.32
GWA-277	189,638	2,565,966	103.23	12/05/03	99.90	3.10
GWA-278-SUPPLY	112,821	2,552,013	95.73	11/30/03	93.46	2.54
GWA-279	146,628	2,614,918	93.60	12/05/03	93.46	0.54
GWA-286A	127,242	2,555,111	98.13	11/29/03	96.68	1.32
GWA-287	174,779	2,582,911	105.54	12/05/03	103.13	2.87
GWA-289	115,313	2,566,701	99.00	11/29/03	96.68	2.32
GWA-291	178,481	2,589,837	105.95	12/01/03	103.13	2.87
GWA-292	93,360	2,575,109	94.41	11/29/03	93.46	0.54
GWA-294	118,813	2,570,303	101.07	12/05/03	99.90	1.10
GWA-296	218,425	2,546,138	91.60	12/05/03	90.24	1.76
GWA-297	88,640	2,574,511	92.54	11/29/03	93.46	-0.46
GWA-301	83,390	2,574,920	90.50	11/29/03	90.24	0.76
GWA-302	128,694	2,561,669	100.67	12/07/03	96.68	4.32
GWA-305	154,574	2,592,522	105.12	12/05/03	103.13	1.87
GWA-307	219,321	2,554,853	90.97	12/08/03	93.46	-2.46
GWA-310	214,549	2,555,577	98.19	12/05/03	93.46	4.54
GWA-317-SUPPLY	164,266	2,587,918	104.00	12/01/03	103.13	0.87
GWA-320	162,538	2,580,321	106.98	12/05/03	103.13	3.87
GWA-328	160,417	2,583,232	106.86	12/05/03	103.13	3.87
GWA-334	192,140	2,583,875	105.62	12/05/03	103.13	2.87
GWA-337	195,909	2,575,513	105.77	12/05/03	103.13	2.87
GWA-338	194,030	2,579,972	106.21	12/06/03	103.13	2.87
GWA-339	125,458	2,556,399	98.52	11/29/03	96.68	2.32
GWA-341	142,376	2,587,076	105.67	12/01/03	103.13	2.87
GWA-343	204,005	2,582,047	105.33	12/05/03	103.13	1.87
GWA-344	119,032	2,557,966	98.19	11/30/03	96.68	1.32
GWA-345	89,167	2,610,584	87.42	12/05/03	87.01	-0.01
GWA-346	115,307	2,555,136	96.96	11/30/03	96.68	0.32
GWA-348	112,031	2,576,321	100.20	11/29/03	96.68	3.32
GWA-349	116,925	2,576,201	101.42	11/29/03	99.90	1.10
GWA-350	79,477	2,605,747	75.07	12/05/03	83.79	-8.79
GWA-352	121,741	2,573,820	102.36	11/29/03	99.90	2.10
GWA-357	232,950	2,539,914	90.03	12/08/03	87.01	2.99
GWA-358	180,600	2,604,992	100.72	12/01/03	99.90	1.10
GWA-360	129,978	2,567,966	102.57	11/30/03	99.90	3.10

GWA-364	230,775	2,546,927	93.38	12/08/03	90.24	2.76
GWA-365	135,435	2,568,640	103.24	11/30/03	99.90	3.10
GWA-366	175,436	2,594,016	104.22	12/05/03	103.13	0.87
GWA-368	140,409	2,569,103	104.00	11/30/03	99.90	4.10
GWA-370	176,595	2,601,551	101.81	12/01/03	103.13	-1.13
GWA-373	145,116	2,570,155	104.29	11/30/03	99.90	4.10
GWA-374	176,917	2,586,205	105.57	12/01/03	103.13	2.87
GWA-379	184,491	2,581,318	106.28	12/06/03	103.13	2.87
GWA-380	91,635	2,559,260	92.75	11/29/03	93.46	-0.46
GWA-382	227,016	2,544,188	91.55	12/29/03	90.24	1.76
GWA-387	240,685	2,530,992	85.38	12/05/03	80.57	4.43
GWA-391	237,519	2,536,434	88.99	12/07/03	83.79	5.21
GWA-393	99,771	2,551,959	93.66	11/30/03	93.46	0.54
GWA-402	282,304	2,538,968	92.50	12/05/03	90.24	2.76
GWA-411	102,469	2,593,499	96.24	11/29/03	93.46	2.54
GWA-412	138,026	2,553,462	95.25	12/05/03	93.46	1.54
GWA-415	99,607	2,559,211	94.73	11/29/03	93.46	1.54
GWA-422	85,663	2,568,496	91.34	11/29/03	93.46	-2.46
GWA-427	93,011	2,566,651	93.73	11/29/03	93.46	0.54
GOW-53	92,495	2,630,399	69.88		67.68	2.32
GOW-57	206,400	2,635,787	70.75		70.90	0.10
GOW-85	74,184	2,629,866	66.09		64.45	1.55
GOW-88	216,417	2,618,441	96.07		90.24	5.76
GWA-417	56,922	2,606,796	72.68		70.90	2.10
GWA-435	103,663	2,615,051	89.30		87.01	1.99
GWA-276-SUPPLY	123,088	2,555,179	97.22	11/30/03	96.68	0.32
GWA-226	85,666	2,596,902	90.27	11/29/03	90.24	-0.24
GWA-227	182,355	2,571,356	106.31	12/06/03	99.90	6.10
GWA-228	130,941	2,557,627	99.29	11/30/03	96.68	2.32
GWA-227	182,355	2,571,356	106.31	12/06/03	99.90	6.10
GWA-228	130,941	2,557,627	99.29	11/30/03	96.68	2.32
GWA-396	279,652	2,543,611	92.34	12/05/03	90.24	1.76
GWA-400	239,995	2,540,084	90.92	12/07/03	87.01	3.99
GWA-299	221,386	2,550,290	95.28	12/05/03	90.24	4.76

ملخص الدراسة

شهدت إمارة أبوظبي تطورات سريعة في العقدين الماضيين مما أدى إلى زيادة الضغط على المصادر الطبيعية ومنها مصادر المياه التقليدية المحدودة. المياه السطحية غير متوفره نظراً لندرتها وقلة الامطار التي تسقط بالإضافة إلى الطقس الجاف ودرجات البخر المرتفعة. هذا بالاضافة الى ان معظم المياه الجوفية في الدولة غير متجددة ومالحة نتيجة الضخ الجائرللمياه الجوفية الذي ادى إلى تناقص كبير في مستوى ونوعية المياه الجوفية .

تتم تغطية الطلب على المياه من خلال المياه المحلاه في الامارة . لذا فإن محطات تحلية المياه غالبا ما تعمل تحت مستوى انتاجي ثابت ومصممه لتغطية أقصى الطلب على المياه خلال السنة كما وأن التغيير في الإحتياجات المائية يكون من فصل إلى آخروأحياناً من يوم ألى آخر.ونتجه لذلك يكون هناك فائض من المياه متوفر خلال فتره معينه حيث يكون الطلب قليل على المياه. إن ادارة مصادر المياه والشوؤن البيئية أمران مهمان يتم أخذهما في الإعتبار في إمارة أبوظبي. وللحصول على إداره فعالة لمصادر المياه ولضمان الاستخدام الامثل للمياه المحلاة فان الفائض من محطات التحلية خلال الفترات التي يكون فيها الطلب قليل على المياه يمكن استخدامة لتغذية الخزانات الجوفية والتي من شأنها المحافظه على المياه الجوفية وتحسين انتاجية الخزانات الجوفية . إن دراسة المخاطر البيئية المحتملة والمتوقعة لتخزين المياه امر مهم للغاية حيث انه يجب التأكد من أن الموقع المراد ضخ المياه فيه سيكون بعيد عن الملوثات بمختلف أنواعها ومن مختلف المصادر. ويكون تلوث المياه الجوفية غالباً بسبب مكبات النفايات، تسرب النفط واسباب أخرى مثل حوادث التسرب للمواد الملوثة،المبيدات والأسمدة من المزارع.

الهدف من هذه الدراسة يشمل جزئين رئيسين:-

أولاً: دراسة تغذية المياه الجوفية في الامارة.

ثانيا: تطبيق نموذج عددي لمحاكاة حركة المياه الجوفية وتوقع الأخطار من الملوثات في المنطقه الغربية. وقد تم عمل عدة اختبارات لتقييم حركة الملوثات بناء على عدة افتراضات ـ لتحديد أبعاد وموقع المنطقة الواجب حمايتها في منطقة الشحن الإصطناعي.

جامعة الإمارات العربية المتحدة
عمادة الدراسات العليا
برنامج ماجستير علوم البيئة

تقييم الخطر البيئي على منطقة تغذية المياه الجوفية في ليوا

رسالة مقدمة من الطالبة : انتصار سالم الكثيري

إلى

جامعة الإمارات العربية المتحدة
استكمالاً لمتطلبات الحصول على درجة الماجستير في علوم البيئة